APPENDIX E

GEOLOGY AND SOILS
Land Development Building Geotechnical Report

Auburn Justice Center Geotechnical Report

GEOTECHNICAL ENGINEERING REPORT
for
PLACER COUNTY
LAND DEVELOPMENT BUILDING
Placer County Project No. 4630
Dewitt Center
Auburn, California

Prepared for:
Placer County Department of Facility Services
11476 C Avenue
Auburn, California 95603

Prepared by:
Holdrege & Kull
792 Searls Avenue
Nevada City, California 95959

Project No. 1750-01
November 1, 2002

HOLDREGE & KULL

CONSULTING ENGINEERS • GEOLOGISTS

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Project No. 1750-01
November 26, 2002

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Mr. Doug Hawk
Placer County Department of Facility Services
11476 C Avenue
Auburn, California 95603

Reference: ***Land Development Building***
Placer County Project No. 4630
Dewitt Center
Auburn, California

Subject: ***Dewitt Fault and Seismic Design Criteria***

Dear Mr. Hawk:

This letter amends our geotechnical engineering report dated November 1, 2002 for the Placer County Land Development Building. In Section 2.1 of our report, we stated that the Dewitt Fault appeared to be located in the immediate vicinity of the project site. In Section 4 of our report, we concluded that additional information should be provided regarding the fault with respect to the proposed improvements. We subsequently reviewed a draft *Geological Resources Existing Conditions Report* (URS, February 14, 2002), which addresses the Dewitt Fault with respect to the Dewitt Center site. The URS draft report included a review of documents that we have found to be pertinent to the Dewitt Fault and subject site; thus we anticipate that further research performed by our office would not be warranted. For additional information regarding the Dewitt Fault, please refer to the URS draft report.

The seismic design criteria presented in our geotechnical engineering report were based on 1998 California Building Code and Seismic Source Type C. The design criteria presented in our report also meet the requirements of the 2001 California Building Code.

We appreciate the opportunity to provide geotechnical engineering services for your project. Please contact us if you need any additional information.

Sincerely,

HOLDREGE & KULL

REGISTERED PROFESSIONAL ENGINEER
JASON W. MUIR
No. 60167
Exp. 6/04
CIVIL
STATE OF CALIFORNIA

Jason Muir
C.E. 60167

copy: Dennis Salter, Placer County Department of Facility Services

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HOLDREGE & KULL
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FACILITY SERVICES
2003 JUN -2 PM 12:52

Project No. 1750-01
May 30, 2003

Mr. Doug Hawk
Placer County Department of Facility Services
11476 C Avenue
Auburn, California 95603

Reference: ***Land Development Building***
Placer County Project No. 4630
Dewitt Center
Auburn, California

Subject: ***Revision of Geotechnical Engineering Report***

Dear Mr. Hawk,

Attached are revisions to Table 5.2.6.1 of our *Geotechnical Engineering Report for Placer County Land Development Building* dated November 1, 2002. The original pages 26 and 27 of the report should be replaced with the attached pages. Several of the recommended asphalt concrete and baserock thicknesses were inadvertently transposed in the original table.

Please contact us if you have any questions.

Sincerely,

HOLDREGE & KULL

REGISTERED PROFESSIONAL ENGINEER
JASON W. MUIR
No. 60167
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STATE OF CALIFORNIA

Jason Muir
C.E. 60167

attachment: Table 5.2.6.1 - Alternate Equivalent Pavement Sections

copies: 3 of attachments to Doug Hawk / Placer County Department of Facility Services

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rock drain, provided the collected water is channeled away from the wall. If a geosynthetic blanket is used, backfill must be compacted carefully so that equipment or soil does not tear or crush the drainage blanket.

5.2.6 Pavement Design

Our R-value (ASTM D301) test results of a composite soil sample collected from exploratory trenches/borings T-1, T-4, T-5, T-6, T-7 and B-8 indicated that the soil had an R-value of 26 by exudation pressure. R-value calculated by expansion pressure was 17, 19 and 21 for TIs of 4, 5 and 6, respectively. Recommended pavement sections for TIs of 4, 5 and 6 are presented in the following table. Some of the section thicknesses presented below may not meet minimum section thicknesses required by the local building official. Compaction requirements are based on compaction relative to the maximum dry density per ASTM D 1557, Modified Proctor.

Table 5.2.6.1 - Alternate Equivalent Pavement Sections
Placer County Land Development Building

Traffic Index: 4 Design R-Value: 17	Pavement Section Alternate A (feet)	Pavement Section Alternate B (feet)
Caltrans Section 26, Standard Specifications, Asphalt Concrete	0.20	---
Caltrans Section 26, Class 2 Baserock 95% compaction	0.50	---
Subgrade Soil 95% compaction	0.50	0.50
Traffic Index: 5 Design R-Value: 19	Pavement Section Alternate A (feet)	Pavement Section Alternate B (feet)
Caltrans Section 26, Standard Specifications, Asphalt Concrete	0.20	0.25
Caltrans Section 26, Class 2 Baserock 95% compaction	0.70	0.60
Subgrade Soil 95% compaction	0.50	0.50

Table 5.2.6.1 - Alternate Equivalent Pavement Sections Placer County Land Development Building		
Traffic Index: 6 Design R-Value: 21	Pavement Section Alternate A (feet)	Pavement Section Alternate B (feet)
Caltrans Section 26, Standard Specifications, Asphalt Concrete	0.25	0.30
Caltrans Section 26, Class 2 Baserock 95% compaction	0.85	0.75
Subgrade Soil 95% compaction	0.50	0.50

The upper 6 inches of native soil should be scarified and recompacted to a minimum of 95 percent of the maximum dry density per ASTM D1557 or CTM 216. The upper 12 inches of imported granular fill, if used, and all baserock must also be compacted to a minimum of 95 percent. Subgrade and baserock density must be tested by a representative of H&K. Subgrade must be proof rolled under the observation of a representative of H&K prior to baserock placement.

Steel reinforced concrete slabs should be considered for use in loading bays, service docks, garbage facilities, or other areas where frequent, heavy vehicle loads are anticipated. The project structural engineer should determine slab thickness and steel reinforcement.

Because expansive clay soil was encountered at a depth of 2 to 3 feet bgs near exploratory trenches T-3 and T-4, we recommend that concrete curbs adjacent to landscape areas be extended a minimum of 12 inches below finish subgrade elevation to reduce surface water infiltration and seasonal moisture variation beneath the pavement.

6 LIMITATIONS

The following limitations apply to the findings, conclusions and recommendations presented in this report:

HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS

Project No. 1750-01
November 1, 2002

Mr. Doug Hawk
Placer County Department of Facility Services
11476 C Avenue
Auburn, California 95603

Reference: ***Land Development Building***
Placer County Project No. 4630
Dewitt Center
Auburn, California

Subject: ***Geotechnical Engineering Report***

Dear Mr. Hawk,

This report presents the results of our geotechnical engineering investigation for the proposed Land Development Building to be located at the Dewitt Center in Auburn, California. We understand that the currently proposed building footprint measures roughly 50,000 to 60,000 square feet and the proposed paved parking areas will cover approximately 185,000 square feet of the approximately 10-acre site.

The findings and recommendations presented in this report are based on our subsurface investigation, laboratory test results, engineering analysis, and our experience with subsurface conditions in the area. Our opinion is that the project can be completed as proposed, provided the recommendations presented in this report are implemented. Our primary concerns, from a geotechnical engineering standpoint, are relatively shallow, resistant rock and expansive clay soil in portions of the site. We should be allowed to perform testing and observation services during grading to confirm our recommendations.

Please contact us if you have any questions regarding our observations or the recommendations presented in this report.

Sincerely,

HOLDREGE & KULL

Prepared by: REGISTERED PROFESSIONAL ENGINEER JASON W. MUIR No. 60167 Exp. 6-04 CIVIL STATE OF CALIFORNIA

Jason Muir
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Reviewed by: REGISTERED PROFESSIONAL ENGINEER CHARLES R. KULL No. 2359 Expires 6/03 GEOTECHNICAL STATE OF CALIFORNIA

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APPENDICES

1 INTRODUCTION

At the request of Mr. Doug Hawk of the Placer County Department of Facility Services, Holdrege & Kull (H&K) performed a geotechnical investigation at the proposed Placer County Land Development Building project site in the Dewitt Center in Auburn, California. The geotechnical investigation was performed consistent with the scope of services presented in our proposal for the project dated August 23, 2002 (revised August 29, 2002), a copy of which is included as Appendix A of this report. For your review, Appendix B contains a document prepared by ASFE entitled *Important Information About Your Geotechnical Engineering Report*, which summarizes the general limitations, responsibilities, and use of geotechnical reports.

1.1 SITE DESCRIPTION

The approximately 10-acre Land Development Center project site is located on the north side of the Dewitt Center in Auburn, California. The site is bordered by Bell Road to the north, Richardson Drive to the west, the Placer County administrative center to the south, and the sheriff's department to the east. The site is bisected from east to west by A Avenue, and East Drive and West Drive are located in the northeast portion of the site.

The northern portion of the site, between Bell Road and A Avenue, is generally comprised of an open lawn area. The southern portion of the site contains four partially occupied residential buildings and associated parking and landscape areas. The Sheriff's department and associated parking and storage buildings occupy the southeast portion of the site.

1.2 PROPOSED IMPROVEMENTS

Our understanding of the proposed improvements are based on our conversations with Doug Hawk of the Placer County Department of Facility Services and our review of a conceptual site plan prepared by Williams + Paddon, Architects (July 19, 2002). We understand that the proposed improvements may include the construction of a Land Development Building which will cover roughly 50,000 to 60,000 square feet of the northwest portion of the site, and roughly 185,000 square feet of paved parking area on the southern portion of the site. A plaza is to be located immediately southeast of the proposed building, and landscape areas will surround the building and extend along the eastern boundary of the site. We anticipate that grading associated with the project will include scarification and recompaction of near surface soil, but relatively minor cut and fill.

1.3 PURPOSE

We performed a geotechnical engineering investigation at the site, collected soil samples for laboratory testing, and performed engineering calculations to provide foundation design criteria, grading and drainage recommendations, and pavement design for the project.

1.4 SCOPE OF SERVICES

To prepare this report, we performed the following scope of services:

- We performed a site investigation, including a literature review and a limited subsurface investigation.

- We collected relatively undisturbed soil samples and bulk soil samples from selected exploratory trenches.

- We performed laboratory tests on select soil samples obtained during our subsurface investigation to determine their engineering material properties.

- Based on observations made during our subsurface investigation and the results of laboratory testing, we performed engineering calculations to provide foundation design criteria, grading and drainage recommendations, and pavement design for the project.

2 SITE INVESTIGATION

We performed a site investigation to characterize the existing site conditions and to develop geotechnical engineering recommendations and design criteria for earthwork and structural improvements. Our site investigation included a literature review and field investigation as described below.

2.1 LITERATURE REVIEW

As a part of our site investigations, we reviewed the Geologic Map of the Sacramento Quadrangle published by the California Department of Conservation, Division of Mines and Geology. The geologic map indicated that the project site is underlain by Paleozoic aged metavolcanic rock. The Paleozoic era spans the period of time between 230 and 600 million years before present (MYBP).

According to the *Soil Survey of Placer County, California, Western Part* (Soil Survey) (United States Department of Agriculture Soil Conservation Service and issued July 1980), the soil class associated with the project site is the Auburn silt loam. This soil type is described as a shallow, residually formed, undulating, well drained soil underlain by vertically tilted metamorphic rock. The typical surface layer is strong brown silt loam extending to an approximate depth of 4 inches below the ground surface (bgs). The surface layer is underlain by yellowish red silt loam. Basic schist is typically encountered at a depth of 20 inches bgs. The soil survey describes the soil as having severe limitations to building development due to the relatively shallow depth to resistant rock.

We reviewed the Fault Activity Map of California and Adjacent Areas published by the California Department of Conservation Division of Mines and Geology (CDMG) in 1994. The fault activity map indicated that the Dewitt Fault was located in the immediate vicinity of the project site. A portion of the Dewitt Fault was described by the fault activity map as having shown quaternary displacement (during the past 1.6 million years) based on geomorphic evidence. We were not able to determine the exact location of the Dewitt Fault with respect to the proposed improvements due to the scale of the fault activity map.

We also reviewed the General Geology of the Auburn 15-Minute Quadrangle, which is included in the *Mineral Land Classification of the Auburn 15' Minute Quadrangle* published by CDMG in 1984. The general geology map indicates a fault, in the apparent alignment of the Dewitt Fault, passes near the proposed building location.

Additional research is presently being conducted to locate the Dewitt Fault with respect to the project site and will be presented as an amendment to this report.

2.2 FIELD INVESTIGATION

We performed our field investigation on October 18 and 22, 2002. During our field investigation, we observed the local topography and general surface conditions and performed a subsurface investigation. The surface and subsurface conditions observed during our field investigation are summarized in the following sections.

Our subsurface investigation included the excavation of seven exploratory trenches across the project site, at the approximate locations shown on Figure 1. In addition, we advanced one hand-augered boring on the east side of the site. The hand-augered boring was used to obtain a bulk soil sample for R-value testing with minimum soil disturbance. We excavated to depths ranging from 15 inches to 4.5 feet using a Case 580 backhoe equipped with a 24-inch bucket. An engineer from our firm logged the soil conditions revealed in the exploratory trenches and collected relatively undisturbed and bulk soil samples for laboratory testing.

2.2.1 Surface Conditions

At the time of our investigation, the project site was observed to be relatively flat lying. The northern portion of the site was covered by lawn. Paved parking areas and A Avenue bisected the central portion of the site from east to west. Four residential buildings, which were surrounded by paved parking areas and landscape areas, were located south of A Avenue. Buildings and paved parking areas associated with the sheriff's department covered much of the southeast portion of the site.

2.2.2 Subsurface Soil Conditions

The soil conditions described in the following paragraphs are generalized, based on our observations of soil revealed in our seven exploratory trenches and one hand-augered boring. More detailed information can be found in the trench logs in Appendix C.

Our exploratory trenches revealed that near-surface soil across much of the site was dark brown to red-brown, loose to medium dense, silty sand and sandy silt with minor clay content and common fine roots.

The near-surface soil was underlain at some trench locations by orange-brown, medium dense, sandy silt with minor clay at a depth of approximately 1.5 feet bgs.

Olive, firm, expansive clay was encountered in exploratory trenches T-3 and T-4 at a depth of 2 to 3 feet bgs.

Severely to moderately weathered, highly fractured, metamorphic rock was generally encountered at depths ranging from 1 to 3 feet bgs. Near refusal of the backhoe was encountered at depths ranging from 15 inches (T-7) to 4.5 feet (T-3) bgs in moderately weathered metavolcanic rock.

Trench T-1 intercepted the alignment of a deep sanitary sewer trench, revealing a portion of the near-surface sewer trench backfill. Based on the relatively shallow depth of resistant rock in the area, blasting was likely necessary for deep excavation along the trench alignment. The sewer trench appeared to have been backfilled with native material. The backfill was described as light brown, medium dense, silty sand with minor clay content, abundant gravel, and angular rock to 12 inches in diameter. Collection of relatively undisturbed samples of the backfill was generally not feasible due to the high rock content.

2.2.3 Groundwater Conditions

During our site investigation, we did not observe seepage in the sidewalls of exploratory trenches, nor did we encounter groundwater in our exploratory trenches. Our investigation was performed at the end of the dry season.

3 LABORATORY TESTING

We performed laboratory tests on selected soil samples collected from our exploratory trenches to determine their engineering properties. Laboratory test results were used to provide geotechnical engineering recommendations and design criteria for the proposed improvements. We performed the following laboratory tests:

- Moisture Content,
- Density (unit weight),
- Direct Shear Strength,

- Expansion Index,
- Atterberg Limits,
- Compaction Curve,
- Resistance Value (R-Value), and
- a suite of landscape gardening soil tests.

Moisture/density and direct shear test results are summarized in Table 3.1 below. Graphical direct shear, expansion index, Atterberg Limits compaction curve, and R-Value test results are presented in Appendix D.

Table 3.1 - Summary of Moisture/Density and Direct Shear Testing

Trench Number	Sample Number	Depth (feet)	Dry Density (pcf)	Moisture Content (%)	Shear Friction Angle (degrees)	Shear Cohesion (psf)
T-1	BT 1-1	1.0	126.7	8.8	--	--
T-2	BT 2-1	1.5	87.7	12.0	31	131
T-2	BT 2-2	2.0	82.9	8.9	--	--
T-3	BT 3-1	1.5	85.8	10.5	--	--
T-4	BT 4-1	1.5	86.6	12.0	--	--
T-5	BT 5-1	1.5	97.9	14.7	--	--

Atterberg limits determination for sample CB 3-2, described as olive clay and obtained from a depth of 2.5 to 3 feet in exploratory trench T-3, indicated that the portion of the sample passing the No. 40 sieve had a liquid limit of 41, a plastic limit of 18, and a plasticity index of 23. The sample was described as a low plasticity clay.

We also performed expansion index testing in general accordance with UBC guidelines using bulk soil sample CB 3-2. A portion of the sample was remolded in general accordance with UBC guidelines and submerged in water under an applied loading of 144 pounds per square foot (psf). The expansion index corrected to 50 percent saturation was 110. Sample CB 3-2 exhibited high expansion potential as classified by UBC guidelines.

Compaction curve testing for bulk soil sample CB 1-1, obtained from utility trench backfill at a depth of 0.5 to 2.0 feet in exploratory trench T-1, resulted in a maximum dry density of 137.0 pounds per cubic feet (pcf) and an optimum moisture content of 8.7 percent, per ASTM D1557 guidelines.

R-value testing was performed for composite soil sample COMP-1, described as red-brown, fine sandy silt with clay. Testing resulted in an R-value of 26. R-value by expansion pressure was determined to be 17, 19 and 21 for traffic indices (TIs) of 4, 5 and 6, respectively. Sample COMP-1 was composed of portions of samples CB 1-1, CB 4-1, CB 5-1, CB 6-1, CB 7-1 and CB 8-1.

We performed a suite of landscape gardening soil tests on composite sample COMP-2. Test results are included in Appendix D. The test package included the following tests: soil saturation percent, soil texture, infiltration rate, pH, conductivity, total dissolved salts, cation exchange capacity, potassium, sodium, calcium, magnesium, nitrate, phosphate, organic matter, sulfate, boron, sulfur or lime requirement, gypsum requirement, sodium absorption ratio, and exchangeable sodium percent. Sample COMP-2 was composed of portions of samples CB 2-1, CB 4-1 and CB 5-1.

4 CONCLUSIONS

The following conclusions are based on our field observations, laboratory test results, and our experience in the area.

- Our opinion is that the site is suitable for the proposed improvements, provided that the geotechnical engineering recommendations and design criteria presented in this report are incorporated into the project plans.

- Prior to grading and construction, we should be allowed to review the proposed grading plan and structural improvements to confirm our recommendations.

- We observed soil/rock conditions to a maximum depth of approximately 4.5 feet bgs. Exploration depth was limited by resistant metamorphic rock. The soil and groundwater conditions below that depth are unknown. The surface soil across the proposed building footprint was generally comprised of native, residual soil underlain at shallow depth by severely to moderately weathered rock.

- Based on our observations of surface and subsurface soil/rock conditions, our primary concern, from a geotechnical standpoint, is presence of relatively shallow, resistant rock and the presence of expansive clay in portions of the site.

- During our subsurface investigation, the Case 580 backhoe met near-refusal at depths ranging from 15 inches (exploratory trench T-7) to 4.5 feet bgs (exploratory trench T-3) in moderately to slightly weathered metavolcanic rock.

- We encountered expansive clay in exploratory trenches T-3 and T-4 at a depth of 2 to 3 feet bgs. The clay soil was generally encountered immediately above weathered metamorphic rock. The low plasticity olive clay obtained from trench T-3 had a liquid limit of 41 and a plasticity index of 23. The sample exhibited high expansion potential as classified by UBC guidelines. Recommendations to reduce the impact of seasonal shrink/swell effects exhibited by the expansive clay are included in the *Grading*, *Foundations*, *Slabs-On-Grade* and *Pavement Design* sections on pages 9, 19, 22 and 26, respectively. We encountered the expansive clay outside of the proposed building footprint. In general, if expansive clay is encountered withing the proposed footprint, the footings should be deepened through expansive clay into the underlying weathered rock, and slabs-on-grade should be designed for the anticipated shrink/swell effects, or the soil should be removed and replaced with predominantly granular material.

- During our investigation, we did not encounter subsurface seepage in our exploratory trenches. We anticipate that seasonal subsurface seepage will be encountered near the surface soil/metamorphic rock interface, particularly during or immediately following the rainy season. In addition, our experience in the region has revealed that groundwater may be perched on rock in relatively level or gently sloping areas well into the summer months. If encountered, perched groundwater may require ripping and air drying of subgrade soil or lime treatment to facilitate grading, even during the summer months. Recommendations pertaining to shallow subsurface seepage are presented in the *Construction Dewatering* section on page 17.

- Trench T-1 intercepted the alignment of a deep sanitary sewer trench, revealing a portion of the near-surface trench backfill. The backfill was described as relatively dense, silty sand with minor clay content, abundant gravel, and angular rock to 12 inches in diameter. Collection of relatively undisturbed samples of the backfill was not feasible due to the high rock content. We recommend that the backfill be observed and tested, if possible, at other locations during site grading to confirm its suitability to support the proposed improvements.

- Additional information regarding the proximity of the Dewitt Fault to the project site will be provided as an addendum to this report. An onsite geologic hazards investigation may be warranted based on the results of our additional research.

5 RECOMMENDATIONS

The following geotechnical engineering recommendations are based on our understanding of the project as currently proposed, our field observations, the results of our laboratory testing program, engineering analysis, and our experience in the area.

5.1 GRADING

As the site was relatively flat-lying at the time of our investigation, we anticipate that the proposed earthwork improvements will involve relatively little cut and fill.

The following sections present our grading recommendations. The grading recommendations address site preparation for fill placement, fill construction, fill slope grading, erosion control, subsurface drainage, surface water drainage, and plan review and construction monitoring.

5.1.1 Clearing and Grubbing

Areas proposed for grading and fill placement should be cleared of vegetation, loose surface soil, and other deleterious materials as described below.

1. Strip and remove the top 1 to 2 inches of soil containing shallow roots and other deleterious materials. Stripped soil, highly organic topsoil or soil containing shallow vegetation, roots and other deleterious materials can be

stockpiled onsite and used in landscape areas, but is not suitable for use as fill.

2. Overexcavate any relatively loose debris and soil that is encountered in our exploratory trenches or any other onsite excavations to underlying, competent material.

3. Overexcavate any loose or untested, existing fill to underlying competent soil, as determined by a representative of H&K.

4. Remove all rocks greater than 6 inches in greatest dimension (oversized rock) from the top 12 inches of soil, if encountered. Oversized rock may be used in landscape areas or removed from the site.

5. Fine grained, potentially expansive soil, as determined by H&K, that is encountered during grading within proposed building locations and paved areas should be mixed with granular soil or overexcavated and stockpiled for removal from the project site or for later use in landscape areas. A typical mixing ratio for granular to expansive soil is 4 to 1. The actual mixing ratio should be determined by H&K.

6. Vegetation, deleterious materials, and oversized rocks not used in landscape areas, drainage channels, or other non-structural uses should be removed from the site.

5.1.2 Cut Slope Grading

Based on our understanding of the project, we do not anticipate that significant cut slopes will be required for the proposed improvements. Cut slopes, if proposed, should be graded with a maximum slope gradient of 2:1, horizontal:vertical (H:V), and should not exceed approximately 8 feet in height. If cut slopes are proposed to be steeper than 2:1, H:V, and/or with a vertical height greater than 4 feet, we should be allowed to review the proposed slope configuration and provide revised recommendations, if appropriate.

5.1.3 Soil Preparation for Fill Placement

After site clearing, the exposed surface soil should be prepared for placement of compacted fill as described below.

1. The surface soil should be scarified to a minimum depth of 8 inches below the existing ground surface and then uniformly moisture conditioned to within approximately 2 percentage points of the ASTM D1557 optimum moisture content.

2. The scarified soil should then be compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density. The moisture content, density and relative percent compaction should be verified by our construction quality assurance (CQA) monitor. The earthwork contractor should assist our CQA monitor by excavating test pads with onsite earth moving equipment.

3. Construction quality assurance tests should be performed using the following minimum testing frequencies, or as determined by the project geotechnical engineer:

Table 5.1.3.1 - Minimum Testing Frequencies for Native Soil Preparation

ASTM No.	Description	Test Frequency
D1557	Modified Proctor Curve	1 per 100,000 sf (1) or material change(2)
D2922	Nuclear Moisture	1 per 10,000 sf
D3017	Nuclear Density	1 per 10,000 sf

Notes: (1) sf = square feet
(2) higher testing frequency shall govern

5.1.4 Fill Placement

Fill placement should incorporate the following recommendations:

1. Soil used for fill construction should consist of uncontaminated, predominantly granular, non-expansive native soil or approved import soil. If encountered, rock used in fill should be broken into pieces no larger than 6 inches in

diameter. Rocks larger than 6 inches are considered oversized material and should be stockpiled for offhaul or later use in landscape areas.

2. Proposed import soil should be predominantly granular, non-expansive and free of deleterious material. Import material that is proposed for use onsite should be submitted to H&K for approval and possible laboratory testing at least 72 hours prior to transport to the site.

3. Cohesive, predominantly fine grained, or potentially expansive soil encountered during grading should be stockpiled for removal, mixed as directed by H&K, or used in landscape areas. We observed highly expansive clay at a depth of 2 to 3 feet bgs in exploratory trenches T-3 and T-4.

4. Soil used to construct fill should be uniformly moisture conditioned to within approximately 2 percentage points of the ASTM D1557 optimum moisture content. Wet soil may need to be air dried or mixed with drier material to facilitate placement and compaction, particularly during or following the wet season.

5. Fill should be constructed by placing uniformly moisture conditioned soil in maximum 8-inch-thick loose, horizontal lifts (layers) prior to compacting.

6. All fill should be compacted to a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density. The upper 8 inches of fill in paved areas should be compacted to a minimum of 95 percent relative compaction.

7. Construction quality assurance tests should be performed using the following minimum testing frequencies, or as determined by the project geotechnical engineer:

Table 5.1.4.1 - Minimum Testing Frequencies for Fill Placement		
ASTM No.	Description	Test Frequency
D1557	Modified Proctor Curve	1 per 3,000 cy (1) or material change(2)
D2922	Nuclear Moisture	1 per 100 cy(3)
D3017	Nuclear Density	1 per 100 cy(3)

Notes: (1) cy = cubic yards
(2) higher testing frequency shall govern
(3) A minimum of 1 test should be taken per every 18 inches of elevation change as fill is placed. Irregular fill or fill of inconsistent quality may require more frequent testing.

The moisture content, density and relative percent compaction of all fill should be verified by our CQA monitor during construction. The earthwork contractor should assist our CQA monitor by excavating test pads with the onsite earth moving equipment.

5.1.5 Differential Fill Depth

The recommendations presented in this section are intended to reduce the magnitude of differential settlement-induced structural distress associated with variable fill depth beneath structures:

1. Site grading should be performed so that cut-fill transition lines do not occur directly beneath any structures. The cut portion of the cut-fill building pads, if proposed, should be scarified to a minimum depth of 12 inches and recompacted to 95 percent relative compaction.

2. Differential fill depths beneath structures should not exceed 5 feet. For example, if the maximum fill depth is 8 feet beneath the proposed building footprint, the minimum fill depth beneath that pad should not be less than 3 feet. If a cut-fill building pad is used in this example, the cut portion would need to be overexcavated 3 feet and rebuilt with compacted fill.

5.1.6 Fill Slope Grading

Based on our understanding of the project, we do not anticipate that significant fill slopes will be required for the proposed improvements. Fill slopes, if proposed should be graded as described below.

1. In general, fill slopes should be no steeper than 2:1, H:V. Proposed fill slope configurations greater than approximately 8 feet in height should be reviewed by H&K. Compaction and fill slope grading must be verified by H&K in the field.

2. Fill should be placed in horizontal lifts to the grades shown on the project plans. Fill slopes should be constructed by overbuilding the slope face and then cutting it back to the design slope gradient. Fill slopes should not be constructed or extended horizontally by placing soil on an existing slope face and/or compacted by track walking.

5.1.7 Erosion Controls

Graded portions of the site should be seeded as soon as possible following grading to allow vegetation to become established prior to and during the rainy season. The following erosion controls should be installed on all cut and fill slopes, if created during grading, to reduce erosion:

1. All slopes created during grading should be hydroseeded or hand seeded/strawed with an appropriate seed mixture compatible with the soil and climate conditions of the site as recommended by the local Resource Conservation District.

2. Following seeding, jute netting should be placed and secured over the slopes to keep seeds and straw from being washed or blown away. Tackifiers or binding agents may be used in lieu of jute netting.

3. Surface water drainage ditches should be established at the top of all slopes to intercept and redirect surface water away from the slope face. Under no circumstances should surface water be allowed to run over slope faces. The

intercepted water should be discharged into natural drainage courses or into other collection and disposal structures.

5.1.8 Underground Utility Trenches

Underground utility trenches should be excavated and backfilled as described below.

1. We anticipate that the contractor will encounter resistant, moderately to slightly weathered rock in excavations as shallow as 1 foot below the existing ground surface in some portions of the site. During our investigation, near refusal of the Case 580K backhoe was encountered at depths ranging from 15 inches (T-7) to 4.5 feet (T-3) bgs in moderately weathered metavolcanic rock. In addition, groundwater seepage should be anticipated in excavations which expose the soil/rock interface.

2. The California Occupational Safety and Health Administration (OSHA) requires all utility trenches deeper than 4 feet bgs to be shored with bracing equipment prior to being entered by any individuals, whether or not they are associated with the project.

3. Utilities should be placed as shallow as possible to reduce the need for blasting, pre-ripping or jack hammering of trenches.

4. We anticipate that shallow subsurface seepage may be encountered, particularly if utility trenches are excavated during the winter, spring, or early summer. The earthwork contractor may need to employ dewatering methods as discussed in the *Construction Dewatering* section on page 17 in order to excavate, place and compact the utility trench backfill materials.

5. Soil used as trench backfill should be non-expansive and should not contain rocks greater than 3 inches in greatest dimension.

6. Soil used to backfill trenches should be uniformly moisture conditioned to within approximately 2 percentage points of the ASTM D1557 optimum moisture content.

7. Trench backfill should be constructed by placing uniformly moisture conditioned soil in maximum 8-inch-thick loose lifts (layers) prior to compacting.

8. Trench backfill placed beneath the utilities (bedding) should be compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density.

9. Trench backfill soil should be compacted to a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density.

10. Trench backfill soil placed within 1 foot of the finished subgrade in road and parking lot areas should be compacted to a minimum relative compaction of 95 percent of the ASTM D1557 maximum dry density.

11. Construction quality assurance tests should be performed during utility trench backfill placement using the following minimum testing frequencies, or as determined by the project geotechnical engineer:

Table 5.1.8.1 - Minimum Testing Frequencies for Trench Backfill

ASTM No.	Description	Test Frequency
D1557	Modified Proctor Curve	1 per 1,000 cy (1) or material change(2)
D2922	Nuclear Moisture	1 per 100 ft trench and 18 inches fill depth(2)
D3017	Nuclear Density	1 per 100 ft trench and 18 inches fill depth(2)

Notes: (1) cy = cubic yards
(2) higher testing frequency shall govern

12. The loose lift thickness, moisture, density and relative compaction of the trench backfill soil should be verified by our CQA Monitor. The earthwork contractor should assist our CQA monitor during construction by excavating test pits in the compacted trench backfill material.

5.1.9 Construction Dewatering

The earthwork contractor should be prepared to dewater excavations if seepage is encountered during grading. Seepage may be encountered if grading is performed during and immediately after the rainy season. In addition, perched groundwater may be encountered on the underlying, resistant metamorphic rock in flat to gently sloping areas even during the summer months. The following recommendations are preliminary and are not based on a groundwater flow analysis. A detailed dewatering analysis was not a part of our proposed scope of services.

1. We anticipate that dewatering of utility trenches can be performed by constructing sumps to depths below the trench bottom and removing the water with sump pumps. Additional sump excavations and pumps should be added as necessary to keep the base of excavations free of standing water when placing and compacting the trench backfill. Because of the relatively level nature of the site, the contractor should not rely on gravity alone to dewater excavations.

2. If seepage is encountered during trench excavation, it may be necessary to remove underlying saturated soil and replace it with free draining, granular drain rock enveloped in geotextile fabric. Native backfill soil can again be used after placing the granular rock to an elevation that is higher than the encountered groundwater.

5.1.10 Subsurface Drainage

Moist or saturated soil conditions will likely be encountered, which limit grading to the drier, summer months. If subsurface seepage or groundwater conditions are encountered which prevent or restrict fill placement, subdrains may be necessary, particularly if grading is performed during or immediately following the wet season. If groundwater or saturated soil conditions are encountered during grading, we should be allowed to observe the conditions and provide site specific subsurface drainage recommendations.

5.1.11 Surface Water Drainage

Proper surface water drainage is important to the successful development of the project. We recommend the following measures to help mitigate surface water drainage problems:

1. Slope final grade in structural areas so that surface water drains away from buildings at a minimum 2 percent slope for a minimum distance of 15 feet.

2. Compact and slope all soil placed adjacent to building foundations such that water is not allowed to pond or infiltrate. Backfill should be free of deleterious material.

3. Direct downspouts to positive drainage or a closed collector pipe which discharges flow to positive drainage.

4. Construct V-ditches at the top of all cut and fill slopes to reduce surface water flow over slope faces. Typically, V-ditches should be 3 feet wide and at least 6 inches deep. Surface water collected in V-ditches should be directed away and downslope from proposed and existing building pads and driveways into a drainage channel.

5.1.12 Grading Plan Review and Construction Monitoring

Construction quality assurance includes review of plans and specifications and performing construction monitoring as described below.

1. We should be allowed to review the final earthwork grading plans prior to construction to confirm our understanding of the project at the time of our investigation, to determine whether our recommendations have been implemented, and to provide additional and/or modified recommendations, if necessary.

2. We should be allowed to perform construction quality assurance and quality control (CQA/QC) monitoring of all earthwork grading performed by the contractor to determine whether our recommendations have been

implemented, and if necessary, provide additional and/or modified recommendations.

5.2 STRUCTURAL IMPROVEMENT DESIGN CRITERIA

5.2.1 Foundations

The following foundation recommendations address foundation construction in competent native soil or fill placed, compacted, and tested in accordance with the recommendations presented in this report.

1. All footings for single story structures should be a minimum of 12 inches wide and trenched through any loose surface material and a minimum of 12 inches into competent native soil or compacted fill placed and tested in accordance with the recommendations presented in this report. Footings for two-story structures should be a minimum of 15 inches wide and trenched through any loose surface material and a minimum of 18 inches into competent native soil or compacted fill placed and tested in accordance with the recommendations presented in this report.

2. If fine grained, potentially expansive soil is encountered at the base of footings, the footing should be deepened through the clay lens into underlying granular soil or weathered rock, as determined in the field by H&K. Highly expansive clay was encountered in exploratory trenches T-3 and T-4 at a depth of 2 to 3 feet below the existing ground surface. Based on our experience in the area, we anticipate that expansive clay may be encountered elsewhere on the site which would require increased footing depth.

3. Footing trenches should be cleaned of all loose soil and construction debris prior to placing concrete. A representative from H&K should observe the footing excavations prior to reinforcing steel and concrete placement.

4. The project structural engineer should design the footings. Minimum steel reinforcement in continuous footings should consist of two No. 4 rebar, one near the top of the footing and one near the bottom. A minimum of 3 inches of concrete coverage should surround the bars.

5. All footings with a minimum embedment depth of 12 inches in competent soil may be sized for an allowable bearing capacity of 2,500 psf for dead plus live loads. This value can be increased by 400 psf for each additional foot of embedment, up to a limiting value of 3,700 psf. Allowable bearing values may be increased by 33 percent for additional transient loading such as wind or seismic.

6. Lateral footing resistance derived from passive earth pressure can be modeled as a triangular pressure distribution ranging from 0 psf at the ground surface to a maximum of 300d psf, where d equals the depth of the footing, in feet.

7. As an alternative to passive resistance, a coefficient of friction of 0.40 between the base of concrete footings and the soil may be used to calculate lateral resistance. Passive pressure and frictional resistance should not be combined when estimating lateral resistance. However, either approach may be considered as an additional factor of safety.

8. Footing excavations should be saturated prior to placing concrete to reduce the risk of problems caused by wicking of moisture from curing concrete.

9. A coefficient of friction for uplift of 150 psf may be used. This value should only be used for short term (wind) loading. Skin friction should be neglected within one foot of the ground surface.

10. Footing excavations should be saturated prior to placing concrete to reduce the risk of problems caused by wicking of moisture from curing concrete.

11. We anticipate that resistant rock may be encountered which limits footing trench excavation. Where footings are proposed to be constructed on competent rock, a higher allowable bearing capacity may be employed as determined by the project geotechnical engineer. Rock anchors are discussed in the following section.

5.2.2 Rock Anchors

Rock anchors or doweling may be used to provide lateral and uplift resistance where shallow, competent rock limits footing excavation. Rock anchors should only be installed in competent rock, to be determined in the field by a representative of H&K. The design of rock anchors should include the following criteria.

1. Pull-out resistance for rock anchors will generally be limited by the shear resistance between the grout and the native rock. For design purposes, a pull-out resistance of 50 pounds per square inch of grout/competent rock contact may be used. Because of the strain in the anchor steel during pull-out, we recommend that the upper 6 inches of grout/competent rock contact be neglected when sizing for uplift.

2. We recommend that the drilled hole have a minimum ½-inch annular clearance between the steel and surrounding rock. Thus, grouting a No. 4 rebar would require a 1½-inch diameter hole.

3. Lateral shear resistance for rock anchors should be designed using V_s=0.45 F_y, where F_y equals the tensile strength of the steel. To develop this shear resistance, a minimum steel embedment of 8 inches into undisturbed, competent rock should be used.

4. The anchor holes should be thoroughly cleaned with compressed air prior to grouting steel.

5. We recommend using a cement grout that has a water/cement ratio of less than 0.6 to construct rock anchors. If high strength epoxy or other adhesives are proposed, H&K should review the proposed rock anchor detail prior to construction.

6. If rock anchors are used on more than 10 percent of the foundation system of any given structure, a representative of H&K should perform pull tests on select anchors.

5.2.3 Seismic Design Criteria

The site is located in Seismic Zone 3 of the 1998 California Building Code (CBC) Seismic Zone Map. CBC seismic design coefficients are listed in Table 5.2.3.1 below.

Table 5.2.3.1 - CBC Seismic Design Coefficients

Seismic Zone Factor, Z(1)	Soil Profile Type(2)	Seismic Coefficient C_a(3)	Seismic Coefficient C_v(4)
0.30	S_B	0.30	0.30

Notes: (1) Table 16-I, 1998 CBC
(2) Table 16-J, 1998 CBC
(3) Table 16-Q, 1998 CBC
(4) Table 16-R, 1998 CBC

Our opinion is that the site may experience moderate ground shaking caused by earthquakes occurring along offsite faults. Earthquakes may cause cracking of concrete slabs, building walls, and pavement at the site.

5.2.4 Slab-on-Grade Floor Systems

A concrete slab-on-grade floor may be used in conjunction with the perimeter concrete foundation. We make the following recommendations regarding the slab-on-grade construction on competent, prepared native soil or compacted fill placed and tested in accordance with the recommendations presented in this report:

1. Slabs-on-grade should be a minimum of 4 inches thick. If floor loads higher than 250 psf, vehicle loads, or intermittent live loads are anticipated, a structural engineer should determine the slab thickness and steel reinforcing schedule.

2. As a minimum, No. 3 rebar on 24-inch centers or flat sheets of 6x6, W2.9 x W2.9 welded wire mesh (WWM) should be used as slab reinforcement. We do not recommend using rolls of WWM because vertically centered placement of rolled mesh within the slab is difficult to achieve. All rebar and sheets of WWM should be placed in the center of the slab and supported on concrete "dobies". We do not recommend "hooking and pulling" of steel during concrete placement.

3. Slabs should be underlain by 4 inches of crushed, washed rock. The rock should be uniformly graded so that 100% passes the 1-inch sieve, with 0% to 5% passing the No. 4 sieve. The rock should be overlain by a vapor barrier at least 10 mils thick. A minimum of 2 inches of clean sand should be spread over the vapor barrier. The sand will act as a leveling pad and aid in curing the concrete. Prior to pouring concrete, the sand leveling pad should be moistened to reduce moisture withdrawal of the concrete during curing.

   The vapor barrier and sand may be omitted in areas that do not have moisture sensitive floor coverings (i.e., garage slabs and parking areas).

4. Regardless of the type of vapor barrier used, moisture can wick up through a concrete slab. Excessive moisture transmission through a slab can cause adhesion loss, warping and peeling of resilient floor coverings, deterioration of adhesive, seam separation, formation of air pockets, mineral deposition beneath flooring, odor and fungi growth. Slabs can be tested for water transmissivity in areas that are moisture sensitive. To further reduce the chance of excessive moisture transmission, a waterproofing consultant can be contacted.

5. Expansion joints should be provided between the slab and perimeter footings and bisect the length and width of the slab at intervals specified by the American Concrete Institute (ACI) or Portland Concrete Association (PCA).

6. Exterior slabs-on-grade such as sidewalks may be placed directly on compacted fill without the use of a baserock section. For exterior slabs, the native soil should be ripped to a depth of 8 inches, moisture conditioned and recompacted. To reduce the likelihood of vertical movement, exterior slabs should not be constructed on potentially expansive soil.

7. Soil should be moisture conditioned prior to placing concrete. If the soil is not moisture conditioned prior to placing concrete, moisture will be wicked out of the concrete, possibly causing shrinkage cracks. Additionally, our opinion is that the moisture conditioning the soil prior to placing concrete will reduce the likelihood of soil swell or heave following construction.

8. All deleterious material must be removed prior to placing slab concrete.

9. Exposed concrete slabs should be moisture cured for at least seven days after placement.

10. Concrete slabs impart a relatively small load on the subgrade (approximately 50 psf). Therefore, some vertical movement should be anticipated from possible expansion or differential loading. In areas where vertical movement must be minimized, the subgrade soil should be tested for swell potential. Potentially expansive soil encountered at proposed slab locations should be overexcavated and replaced with predominantly granular, non-expansive compacted fill.

5.2.5 Retaining Wall Design Criteria

Based on our understanding of the proposed improvements, we do not anticipate that significant cut or fill will be retained. The recommendations included in this section are provided in the case that retaining walls are employed. The following active and passive pressures are for retaining walls in cut native soil or backfilled with granular onsite soil. If import soil is used, a representative from our firm should be allowed to observe and test the soil to determine its strength properties. The pressures exerted against retaining walls may be assumed to be equal to a fluid of equivalent unit weight.

Table 5.2.5.1 presents equivalent fluid unit weights for cut native soil and onsite fill compacted per the grading recommendations presented in this report. We assume that the retained surface will be no steeper than 5% for a minimum distance of the wall height from the back of the retaining wall. If surcharge loads (such as adjacent building foundations) or live loads will be applied within a distance of the wall height from the back of the wall, we should be allowed to review the loading conditions and revise our recommendations, if necessary.

Transition from an at-rest soil pressure condition to an active soil pressure condition behind a retaining wall generally requires slight yielding of the wall. Literature suggests that this yielding may result in lateral movement of the top of the wall of up to approximately two percent of the wall height. Therefore, some wall movement should be expected during and shortly after construction. If that amount of wall movement is not acceptable, we recommend that the wall be designed using higher equivalent fluid pressures, such as the at-rest fluid pressure.

Table 5.2.5.1 - Equivalent Fluid Unit Weights [(1)]	
Loading Condition	Retained Cut or Compacted Fill (see grading recommendations)
Active Pressure (pcf)	30
Passive Pressure (pcf)	300
At-Rest Pressure (pcf)	45
Coefficient of Friction	0.40

Note: (1) The equivalent fluid unit weights presented are ultimate values and do not include a factor of safety. The passive pressures provided assume footings are founded in competent native soil or engineered fill.

Recommendations for design and construction of retaining walls are listed below:

1. Compaction equipment should not be used directly adjacent to retaining walls unless the wall is designed or braced to resist the additional lateral pressures.

2. If any surface loads are closer to the top of the retaining wall than the height of the wall, H&K should review the loads and loading configuration. We should be allowed to review wall details and plans for any wall over 10 feet in height.

3. All retaining walls must be well drained to reduce hydrostatic pressures. Walls should be provided with a drainage blanket to reduce additional lateral forces and minimize saturation of the backfill soil. Drainage blankets may consist of graded rock drains or geosynthetic blankets.

4. Rock drains should consist of a minimum 12-inch wide, Caltrans Class II, permeable drainage blanket, placed directly behind the wall; or crushed washed rock enveloped in a non-woven geotextile filter fabric such as Amoco 4546™ or equivalent. Drains should have a minimum 4-inch diameter, perforated, schedule 40, PVC pipe placed at the base of the wall, inside the drainrock, with the perforations placed down. The PVC pipe should be sloped so that water is directed away from the wall by gravity. A geosynthetic drainage blanket such as Enkadrain™ or equivalent may be substituted for the

rock drain, provided the collected water is channeled away from the wall. If a geosynthetic blanket is used, backfill must be compacted carefully so that equipment or soil does not tear or crush the drainage blanket.

5.2.6 Pavement Design

Our R-value (ASTM D301) test results of a composite soil sample collected from exploratory trenches/borings T-1, T-4, T-5, T-6, T-7 and B-8 indicated that the soil had an R-value of 26 by exudation pressure. R-value calculated by expansion pressure was 17, 19 and 21 for TIs of 4, 5 and 6, respectively. Recommended pavement sections for TIs of 4, 5 and 6 are presented in the following table. Some of the section thicknesses presented below may not meet minimum section thicknesses required by the local building official. Compaction requirements are based on compaction relative to the maximum dry density per ASTM D 1557, Modified Proctor.

Table 5.2.6.1 - Alternate Equivalent Pavement Sections
Placer County Land Development Building

Traffic Index: 4 Design R-Value: 17	Pavement Section Alternate A (feet)	Pavement Section Alternate B (feet)
Caltrans Section 26, Standard Specifications, Asphalt Concrete	0.20	0.50
Caltrans Section 26, Class 2 Baserock 95% compaction	0.20	0.40
Subgrade Soil 95% compaction	0.50	0.50
Traffic Index: 5 Design R-Value: 19	**Pavement Section Alternate A (feet)**	**Pavement Section Alternate B (feet)**
Caltrans Section 26, Standard Specifications, Asphalt Concrete	0.20	0.70
Caltrans Section 26, Class 2 Baserock 95% compaction	0.25	0.60
Subgrade Soil 95% compaction	0.50	0.50

Table 5.2.6.1 - Alternate Equivalent Pavement Sections Placer County Land Development Building		
Traffic Index: 6 Design R-Value: 21	Pavement Section Alternate A (feet)	Pavement Section Alternate B (feet)
Caltrans Section 26, Standard Specifications, Asphalt Concrete	0.25	0.85
Caltrans Section 26, Class 2 Baserock 95% compaction	0.30	0.75
Subgrade Soil 95% compaction	0.50	0.50

The upper 6 inches of native soil should be scarified and recompacted to a minimum of 95 percent of the maximum dry density per ASTM D1557 or CTM 216. The upper 12 inches of imported granular fill, if used, and all baserock must also be compacted to a minimum of 95 percent. Subgrade and baserock density must be tested by a representative of H&K. Subgrade must be proof rolled under the observation of a representative of H&K prior to baserock placement.

Steel reinforced concrete slabs should be considered for use in loading bays, service docks, garbage facilities, or other areas where frequent, heavy vehicle loads are anticipated. The project structural engineer should determine slab thickness and steel reinforcement.

Because expansive clay soil was encountered at a depth of 2 to 3 feet bgs near exploratory trenches T-3 and T-4, we recommend that concrete curbs adjacent to landscape areas be extended a minimum of 12 inches below finish subgrade elevation to reduce surface water infiltration and seasonal moisture variation beneath the pavement.

6 LIMITATIONS

The following limitations apply to the findings, conclusions and recommendations presented in this report:

1. Our professional services were performed consistent with the generally accepted geotechnical engineering principles and practices employed in northern California. This warranty is in lieu of all other warranties, either expressed or implied.

2. These services were performed consistent with our agreement with our client. We are not responsible for the impacts of any changes in environmental standards, practices, or regulations subsequent to performance of our services. We do not warrant the accuracy of information supplied by others, or the use of segregated portions of this report. This report is solely for the use of our client unless noted otherwise. Any reliance on this report by a third party is at the party's sole risk.

3. If changes are made to the nature or design of the project as described in this report, then the conclusions and recommendations presented in this report should be considered invalid by all parties. Only our firm can determine the validity of the conclusions and recommendations presented in this report. Therefore, we should be allowed to review all project changes and prepare written responses with regards to their impacts on our conclusions and recommendations. However, we may require additional fieldwork and laboratory testing to develop any modifications to our recommendations. Costs to review project changes and perform additional fieldwork and laboratory testing necessary to modify our recommendations is beyond the scope of services presented in this report. Any additional work will be performed only after receipt of an approved scope of services, budget, and written authorization to proceed.

4. The analyses, conclusions and recommendations presented in this report are based on site conditions as they existed at the time we performed our surface and subsurface field investigations. We have assumed that the subsurface soil and groundwater conditions encountered at the location of our exploratory trenches are generally representative of the subsurface conditions throughout the entire project site. However, the actual subsurface conditions at locations between and beyond our exploratory trenches may differ. Therefore, if the subsurface conditions encountered during construction are different than those described in this report, then we

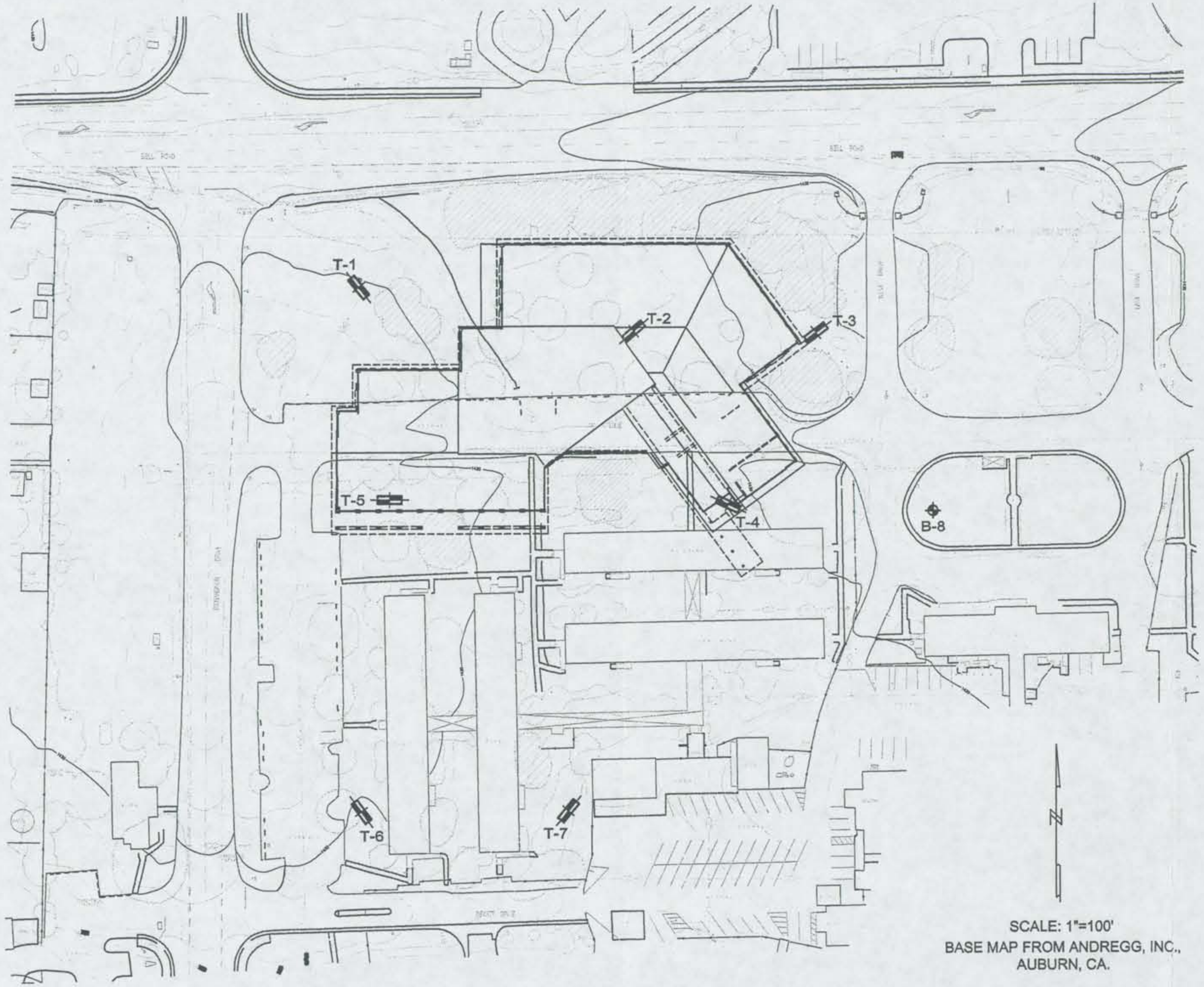
should be notified immediately so that we can review these differences and, if necessary, modify our recommendations.

5. The elevation or depth to the groundwater table underlying the project site may differ with time and location.

6. The project site map shows approximate exploratory trench locations as determined by pacing distances from identifiable site features. Therefore, the trench locations should not be relied upon as being exact nor located with surveying methods.

7. Our geotechnical investigation scope of services did not include evaluating the project site for the presence of hazardous materials. Although we did not observe the presence of hazardous materials at the time of our field investigation, all project personnel should be careful and take the necessary precautions should hazardous materials be encountered during construction.

8. The findings of this report are valid as of the present date. However, changes in the conditions of the property can occur with the passage of time. The changes may be due to natural processes or to the works of man, on the project site or adjacent properties. In addition, changes in applicable or appropriate standards can occur, whether they result from legislation or the broadening of knowledge. Therefore, the recommendations presented in this report should not be relied upon after a period of two years from the issue date without our review.



1750-01_FIG1_SLD

HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS
792 SEARLS AVENUE
NEVADA CITY, CA 95959
(530) 478-1305 FAX 478-1019

APPROXIMATE TRENCH AND BORING LOCATIONS FOR

LAND DEVELOPMENT BUILDING

PLACER COUNTY, CALIFORNIA

DRAWN BY: DFD/SLD	CHECKED BY: JWM
PROJECT NO.: 1750-01	
DATE: OCTOBER, 2002	
FIGURE NO.: 1	

APPENDIX A PROPOSAL

Exhibit "A"

HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS

August 23, 2002
(revised August 29, 2002)

Mr. Doug Hawk
Placer County Department of Facility Services
11476 C Avenue
Auburn, California 95603

Reference: ***Placer County Land Development Building***
Project No. 4630
Dewitt Center, Auburn, California

Subject: ***Proposal for Geotechnical Investigation***

Dear Mr. Hawk,

We appreciate the opportunity to present this proposal to perform a geotechnical investigation for the proposed Land Development Building located at the Dewitt Center in Auburn, California. We prepared this proposal in response to your request for proposals (RFP) dated August 7, 2002. To prepare this proposal, we reviewed the RFP, which included undated, existing and preliminary site plans prepared by Williams + Paddon, Architects + Planners.

We understand that the purpose of our geotechnical investigation will be to provide recommendations for site preparation, grading, foundations and pavement design.

SCOPE OF SERVICES

Geotechnical Investigation

Based on our understanding of the project, we propose to perform the following scope of services:

1. Review geologic maps and soil surveys of the area, including the project site.

(530) 478-1305 • FAX (530) 478-1019 • E-mail: handk@HandK.net • 792 Searls Avenue • Nevada City, CA 95959 • A California Corporation

2. Excavate eight to ten exploratory trenches to maximum depths of 10 feet across the project site. We will collect relatively undisturbed and bulk soil samples from our exploratory trenches for laboratory testing. An engineer or geologist from Holdrege & Kull (H&K) will log the trenches in the field.

3. Perform laboratory tests on select soil samples. Tests will include direct shear, moisture-density determination, compaction curve and R-value. If potentially expansive soil is encountered during our investigation, within the area of proposed improvements, we will perform Atterberg limits tests and swell tests on select samples.

4. Using laboratory test results we will perform the necessary calculations to provide allowable bearing capacities, foundation design criteria, recommended pavement sections, recommendations for site preparation and grading, cut and fill slope gradients, site drainage and slab-on-grade construction.

5. Following completion of the above tasks, we will issue four copies of a geotechnical report which will include:

   a. Logs of exploratory trenches;

   b. Site plan showing approximate locations of our trenches and pertinent geologic features observed during our investigation;

   c. Recommendations for site grading and development, including allowable cut and fill slope gradients and erosion control measures;

   d. Foundation design criteria, including allowable bearing capacities, for the proposed development;

   e. Seismic coefficients;

   f. Recommendations for groundwater and surface water drainage control on the site, if appropriate;

   g. Potential expansion or settlement risks; and

   h. Pavement design.

Alternate Scope of Services

We understand that a geotechnical engineering investigation and report may also be required for the proposed Auburn Justice Center, which is also to be located at the Dewitt Center. Under our alternate scope of services, the scope of services outlined above for the Placer County Land Development Building would also be provided for the Auburn Justice Center.

FEE

Our fee to complete the scope of services described above for the Placer County Land Development Building would be   We would be able to complete geotechnical investigations and reports for both the Auburn Justice Center and the Land Development Building sites, as described above for our alternate scope of services, for a fee of   We understand that the two investigations would be billed as separate projects. We will provide an operated backhoe to perform the investigation(s). Progress billing will be monthly using the attached 2002 fee schedule.

TIMING

We will be able to commence work on the project within one week of receiving authorization to proceed, weather permitting. Our geotechnical report will be submitted within three weeks following completion of field work.

Thank you for the opportunity to provide geotechnical services for the project. Please feel free to contact our office if you have any questions regarding our proposed scope of services.

Sincerely,

HOLDREGE & KULL

Charles R. Kull, G.E., C.E.G.
Principal

attachments: 2002 Fee Schedule

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APPENDIX B *IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL ENGINEERING REPORT*

(included with permission of ASFE, Copyright 1992)

Important Information About Your
Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. *No one except you* should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one—not even you*—should apply the report for any purpose or project except the one originally contemplated.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, *do not rely on a geotechnical engineering report* that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions *only* at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an *opinion* about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject To Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce such risks, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations", many of these provisions indicate where geotechnical engineers responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Rely on Your Geotechnical Engineer for Additional Assistance

Membership in ASFE exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

ASFE PROFESSIONAL FIRMS PRACTICING IN THE GEOSCIENCES

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email: info@asfe.org www.asfe.org

IIGER06983.5M

APPENDIX C EXPLORATORY TRENCH LOGS

TRENCH T-1

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	TRENCH NO.
1750-01	LAND DEVELOPMENT BLDG.	1430 FEET MSL	10/18/02	1 OF 1	T-1

EXCAVATION METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CASE 580 BACKHOE	NONE	NONE	NONE

SAMPLE NO.	BLOW COUNTS (N)	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
					SM	FILL: DARK BROWN, SLIGHTLY MOIST, LOOSE, SILTY SAND WITH COMMON FINE ROOTS
BT 1-1	--	126.7	8.8	1		
				2	SM	FILL: LT. BROWN, SLIGHTLY MOIST, MEDIUM-DENSE SILTY SAND WITH MINOR CLAY CONTENT, ABUNDANT ANGULAR GRAVEL, AND ANGULAR ROCK TO 12" IN DIAMETER
				3		REFUSAL ON MODERATELY TO SLIGHTLY WEATHERED METAVOLCANIC ROCK AT 24 TO 30 INCHES
				4		
				5		
				6		
				7		
				8		
				9		
				10		
				11		
				12		
				13		
				14		
				15		
				16		
				17		
				18		
				19		
				20		

TRENCH T-2

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	TRENCH NO.
1750-01	LAND DEVELOPMENT BLDG.	1429 FEET MSL	10/18/02	1 OF 1	T-2

EXCAVATION METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CASE 580 BACKHOE	SLIDE HAMMER	NONE	NONE

SAMPLE NO.	BLOW COUNTS (N)	DRY DENSITY (PCF)	PERCENT MOISTURE
CB 2-1	--	--	--
BT 2-1	--	87.7	12.0
BT 2-2	--	82.9	8.9

DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
0–1.5	ML	DARK RED-BROWN, SLIGHTLY MOIST, LOOSE, SANDY SILT WITH ABUNDANT FINE ROOTS
1.5–2.5	ML	DARK RED-BROWN, SLIGHTLY MOIST, MEDIUM-DENSE SANDY SILT
2.5–3.5	ML	ORANGE-BROWN, SLIGHTLY MOIST, MEDIUM-DENSE SANDY SILT WITH MINOR CLAY
3.5–5	RX	COMPLETELY TO SEVERELY WEATHERED METAVOLCANIC ROCK
		NEAR REFUSAL ON MODERATELY TO SLIGHTLY WEATHERED METAVOLCANIC ROCK AT 4 FEET

DEPTH scale: 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20

TRENCH T-3

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	TRENCH NO.
1750-01	LAND DEVELOPMENT BLDG.	1431 FEET MSL	10/18/02	1 OF 1	T-3

EXCAVATION METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CASE 580 BACKHOE	SLIDE HAMMER	NONE	NONE

SAMPLE NO.	BLOW COUNTS (N)	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
CB 3-1	--	--	--		ML	DARK RED-BROWN, SLIGHTLY MOIST, LOOSE, SANDY SILT WITH ABUNDANT FINE ROOTS
BT 3-1	--	85.8	10.5	1		
					ML	DARK RED-BROWN, SLIGHTLY MOIST, MEDIUM-DENSE SANDY SILT
BT 3-2	--	--	--	2		
					ML	ORANGE-BROWN, SLIGHTLY MOIST, MEDIUM-DENSE SANDY SILT WITH MINOR CLAY
				3	CL	OLIVE, SLIGHTLY MOIST, FIRM CLAY
				4	SM	COMPLETELY TO SEVERELY WEATHERED METAVOLCANIC ROCK (EXCAVATES AS LIGHT BROWN, SLIGHTLY MOIST, MEDIUM-DENSITY SILTY SAND WITH ANGULAR GRAVEL AND ROCK TO 6" IN DIAMETER)
				5		NEAR REFUSAL ON MODERATELY TO SLIGHTLY WEATHERED METAVOLCANIC ROCK AT 4 FEET B.G.S.
				6		
				7		
				8		
				9		
				10		
				11		
				12		
				13		
				14		
				15		
				16		
				17		
				18		
				19		
				20		

TRENCH T−4

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	TRENCH NO.
1750−01	LAND DEVELOPMENT BLDG.	1430 FEET MSL	10/18/02	1 OF 1	T−4

EXCAVATION METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CASE 580 BACKHOE	SLIDE HAMMER	NONE	NONE

SAMPLE NO.	BLOW COUNTS (N)	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
				0–1	ML	DARK RED−BROWN, SLIGHTLY MOIST, LOOSE, SANDY SILT WITH ABUNDANT FINE ROOTS
BT 4−1	--	86.6	12.0	1–2.5	ML	DARK RED−BROWN, SLIGHTLY MOIST, MEDIUM−DENSITY SANDY SILT
				2.5–3	CL	OLIVE, SLIGHTLY MOIST, FIRM CLAY
				3–4.5	SM	COMPLETELY TO SEVERELY WEATHERED METAVOLCANIC ROCK (EXCAVATES AS LIGHT BROWN, SLIGHTLY MOIST, MEDIUM−DENSITY SILTY SAND WITH ANGULAR GRAVEL AND ROCK TO 6" IN DIAMETER)

Depth scale: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20

TRENCH T-5

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	TRENCH NO.
1750-01	LAND DEVELOPMENT BLDG.	1431 FEET MSL	10/18/02	1 OF 1	T-5

EXCAVATION METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CASE 580 BACKHOE	SLIDE HAMMER	NONE	NONE

SAMPLE NO.	BLOW COUNTS (N)	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
					ML	DARK RED-BROWN, SLIGHTLY MOIST, LOOSE, SANDY SILT WITH ABUNDANT FINE ROOTS
CB 5-1	--	--	--	1	ML	DARK BROWN, SLIGHTLY MOIST, MEDIUM-DENSE SANDY SILT
BT 5-1	--	97.9	14.7	2	SM	COMPLETELY TO SEVERELY WEATHERED METAVOLCANIC ROCK (EXCAVATES AS LIGHT BROWN, SLIGHTLY MOIST, MEDIUM-DENSE SILTY SAND WITH ANGULAR GRAVEL AND ROCK TO 6" IN DIAMETER)
				3		
				4		TRENCH TERMINATED AT 3.5 FEET
				5		
				6		
				7		
				8		
				9		
				10		
				11		
				12		
				13		
				14		
				15		
				16		
				17		
				18		
				19		
				20		

TRENCH T-6

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	TRENCH NO.
1750-01	LAND DEVELOPMENT BLDG.	1430 FEET MSL	10/18/02	1 OF 1	T-6

EXCAVATION METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CASE 580 BACKHOE	HAND	NONE	NONE

SAMPLE NO.	BLOW COUNTS (N)	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
					ML	RED-BROWN, SLIGHTLY MOIST, LOOSE, SANDY SILT WITH ABUNDANT FINE ROOTS
CB 6-1	--	--	--	1	ML	RED-BROWN, SLIGHTLY MOIST, MEDIUM-DENSE SANDY SILT
				2	SM	COMPLETELY TO SEVERELY WEATHERED METAVOLCANIC ROCK (EXCAVATES AS LIGHT BROWN, SLIGHTLY MOIST, MEDIUM-DENSE SILTY SAND WITH ANGULAR GRAVEL AND ROCK TO 6" IN DIAMETER)
				3		TRENCH TERMINATED AT 2.5 FEET
				4		
				5		
				6		
				7		
				8		
				9		
				10		
				11		
				12		
				13		
				14		
				15		
				16		
				17		
				18		
				19		
				20		

TRENCH T-7

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	TRENCH NO.
1750-01	LAND DEVELOPMENT BLDG.	1426 FEET MSL	10/18/02	1 OF 1	T-7

EXCAVATION METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CASE 580 BACKHOE	HAND	NONE	NONE

SAMPLE NO.	BLOW COUNTS (N)	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
					ML	RED-BROWN, DRY, MEDIUM-DENSITY, SANDY SILT WITH MINOR ANGULAR GRAVEL AND TREE ROOTS
CB 7-1	--	--	--	1	SM	MODERATELY TO SLIGHTLY WEATHERED METAVOLCANIC ROCK (EXCAVATES AS LIGHT BROWN, SLIGHTLY MOIST, MEDIUM-DENSE SILTY SAND WITH ANGULAR GRAVEL AND ROCK TO 6" IN DIAMETER)
				2		NEAR REFUSAL OF BACKHOE ON ROCK AT 15 INCHES
				3		
				4		
				5		
				6		
				7		
				8		
				9		
				10		
				11		
				12		
				13		
				14		
				15		
				16		
				17		
				18		
				19		
				20		

BORING B-8

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	BORING NO.
1750-01	LAND DEVELOPMENT BLDG.	1431 FEET MSL	10/18/02	1 OF 1	B-8

EXCAVATION METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
HAND AUGER	HAND	NONE	NONE

SAMPLE NO.	BLOW COUNTS (N)	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
					SM	DARK BROWN, LOOSE SILTY SAND WITH ABUNDANT FINE ROOTS
CB 8-1	--	--	--	1	ML	RED-BROWN, MOIST, MEDIUM-DENSE SANDY SILT WITH CLAY AND MINOR GRAVEL
				2		BORING TERMINATED AT 1.5 FEET
				3		
				4		
				5		
				6		
				7		
				8		
				9		
				10		
				11		
				12		
				13		
				14		
				15		
				16		
				17		
				18		
				19		
				20		

APPENDIX D LABORATORY TEST DATA

Expansion Index/Swell

ASTM D4829

Project No.:	1750-01	Project Name: Land Development Building		Date:	10/24/02
Sample No.:	CB 3-2	Boring/Trench No.: T-3	Depth (ft) 2.5-3.0	Tested By:	JCS
Soil Description:	Olive Clay			Checked By:	MLH
Sample Location:				Lab. No.:	2-553

Specimen Type:	Undisturbed	XX	Disturbed		Remolded to:	
Tube Dia. (inch) =	4.00		Ring Dia. (inch) =	4	Ring Height (inch) =	1.00

FIELD DATA Tube Sample Moisture & Density			LAB DATA	Test wt. 144 Initial	Final	Test wt. Initial	Final	Test wt. Initial	Final
Tare Tube Number			Tare Number	4.00					
Tare Weight	(gr)		Tare Ring Weight (gr)	201.20	201.20				
Wet Soil + Tare	(gr)		Tare Pan Weight (gr)	157.20	157.20				
Dry Soil + Tare	(gr)		Wet Soil + Tare (gr)	712.20	799.13				
Weight of Water	(gr)	0.00	Dry Soil + Tare (gr)	672.21	672.21	0.00		0.00	
Dry Soil Weight	(gr)	0.00	Weight of Water (gr)	39.99	126.92	0.00	0.00	0.00	0.00
Moisture Content	(%)	0.00	Dry Soil Weight (gr)	313.81	313.81	0.00	0.00	0.00	0.00
Soil Height	(in)		Moisture Content (%)	12.74	40.44	0.00	0.00	0.00	0.00
Wet Unit Weight	(pcf)		Wet Unit Weight (pcf)	107.27	119.84				
Dry Unit Weight	(pcf)		Dry Unit Weight (pcf)	95.14	85.33				
			Sample Height (Inches)	1.00	1.115				
Specific gravity		2.68	Percent Saturation	45.08	112.93				

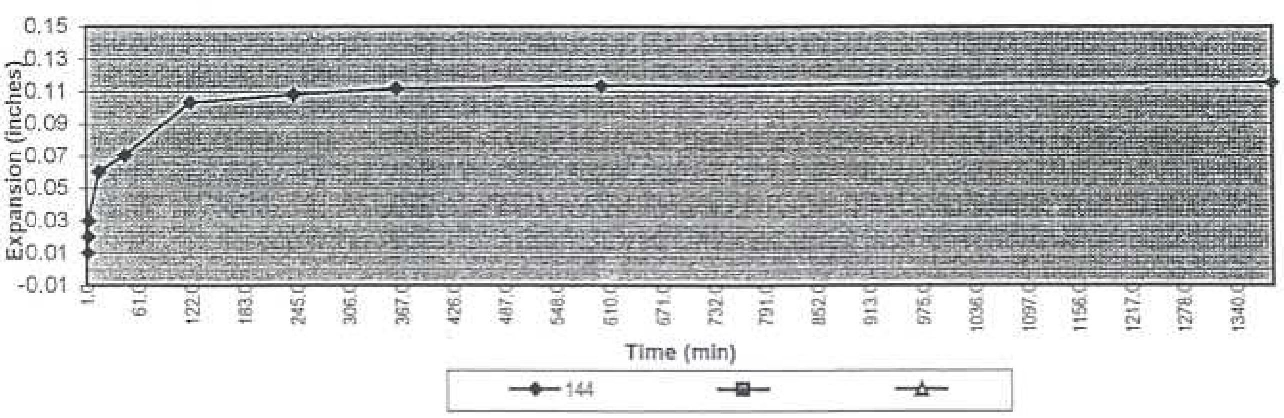
Expanion Index Number

Surcharge (PSF)		Uncorrected	Corrected to 50% Saturation
Test wt.	144	115	110
Test wt.			
Test wt.			

Elapsed Time (m:s)	Change in Height (inches)	Elapsed Time (m:s)	Change in Height (Inches)	Elapsed Time (m:s)	Change in Height (Inches)
1.0	0.0100				
2.0	0.0200				
3.0	0.0300				
15.0	0.0600				
45.0	0.0700				
120.0	0.1030				
240.0	0.1080				
360.0	0.1115				
600.0	0.1130				
1380.0	0.1150				

BC Expansion Index Values and Description	
Expansion Index	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
above 130	Very High

Expansion Versus Time



HOLDREGE & KULL

Atterberg Indices

ASTM D4318

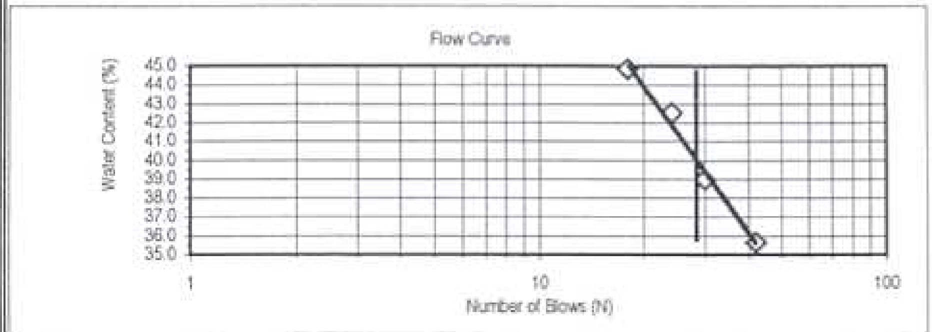
Project No.:	1750-01	Project Name:	Land Development Building			Date:	10/26/02
Sample No.:	CB 3-2	Boring/Trench	T-3	Depth, ft.:	2.5-3.0	Tested By:	JCS
Description:	Olive Clay					Checked By:	MLH
Sample Location:						Lab. No.:	2-553

Estimated % of Sample Retained on No. 40 Sieve: 20 Sample Air Dried: YES

Test Method A or B: A

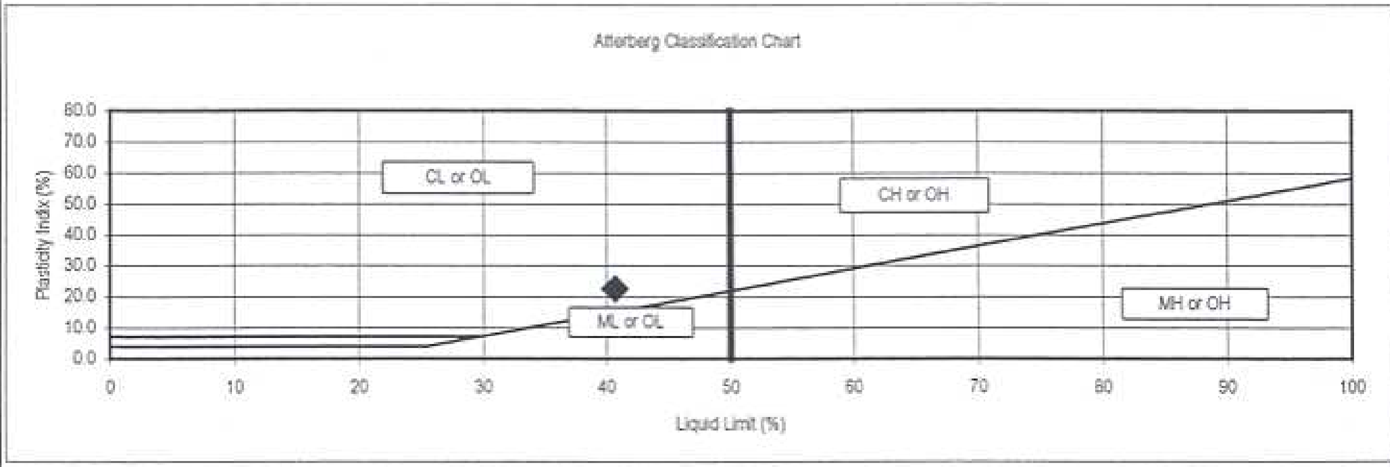
	LIQUID LIMIT:					PLASTIC LIMIT:		
Sample No.:	1	2	3	4	5	1	2	3
Pan ID:	11	20	17	5		1.00	2.00	3.00
Wt. Pan (gr)	92.71	160.47	160.27	100.54		11.26	11.27	4.19
Wt. Wet Soil + Pan (gr)	122.98	185.62	185.48	128.61		16.58	17.70	8.48
Wt. Dry Soil + Pan (gr)	113.95	179.01	178.41	119.92		15.75	16.72	7.82
Wt. Water (gr)	9.03	6.61	7.07	8.69		0.83	0.98	0.66
Wt. Dry Soil (gr)	21.24	18.54	18.14	19.38		4.49	5.45	3.63
Water Content (%)	42.5	35.7	39.0	44.8		18.5	18.0	18.2
Number of Blows, N	24	42	30	18				

LIQUID LIMIT = 40.7 PLASTIC LIMIT = 18.2



Plastic Index = 22.5

Group Symbol = CL



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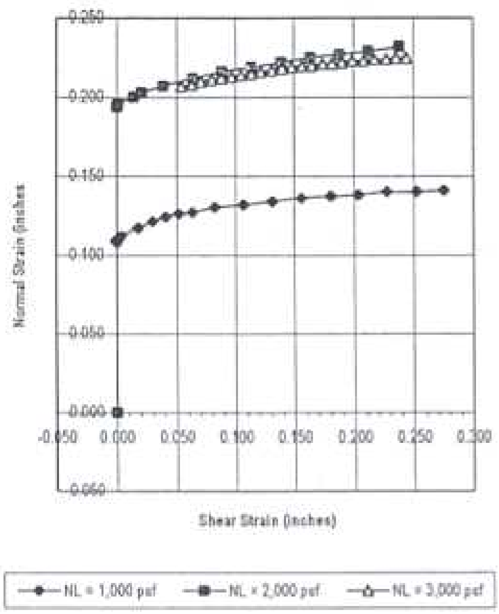
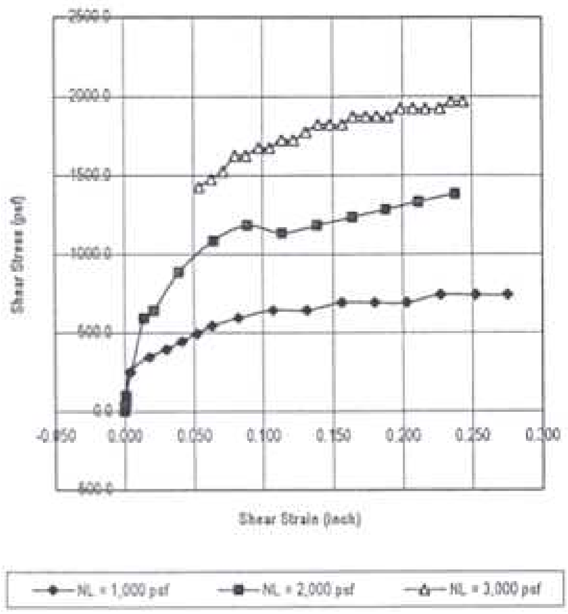
DIRECT SHEAR

TEST RESULTS

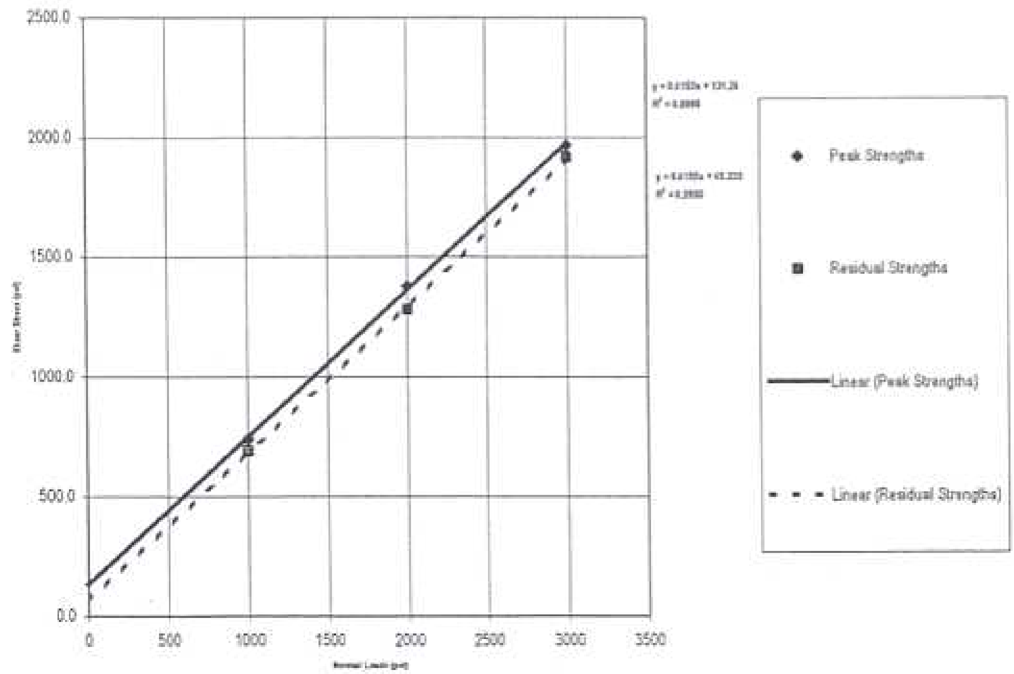
Shear Strain vs. Normal Strain



Shear Strain vs. Shear Stress



Mohr-Columb Failure Envelope



SHEAR STRENGTH TEST RESULTS		
PARAMETERS	PEAK STRENGTH	RESIDUAL STRENGTH:
FRICTION ANGLE, (degree)	31.6	31.6
COHESION, (psf)	131.0	65.0

HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS
792 SEARLS AVENUE
NEVADA CITY, CA 95959
(530) 478-1305 FAX 478-1019

PROJECT NAME:	Land Development Building		
PROJECT NO.:	1750-01	DATE:	10/25/02
BORING / TRENCH NO.:	T-2	LAB NO.	2-553
SAMPLE NO.:	BT 2-1	SAMPLE DEPTH (ft):	1.5
DESCRIPTION:	Orange-Brown Sandy Silt with Clay		

COMPACTION TEST

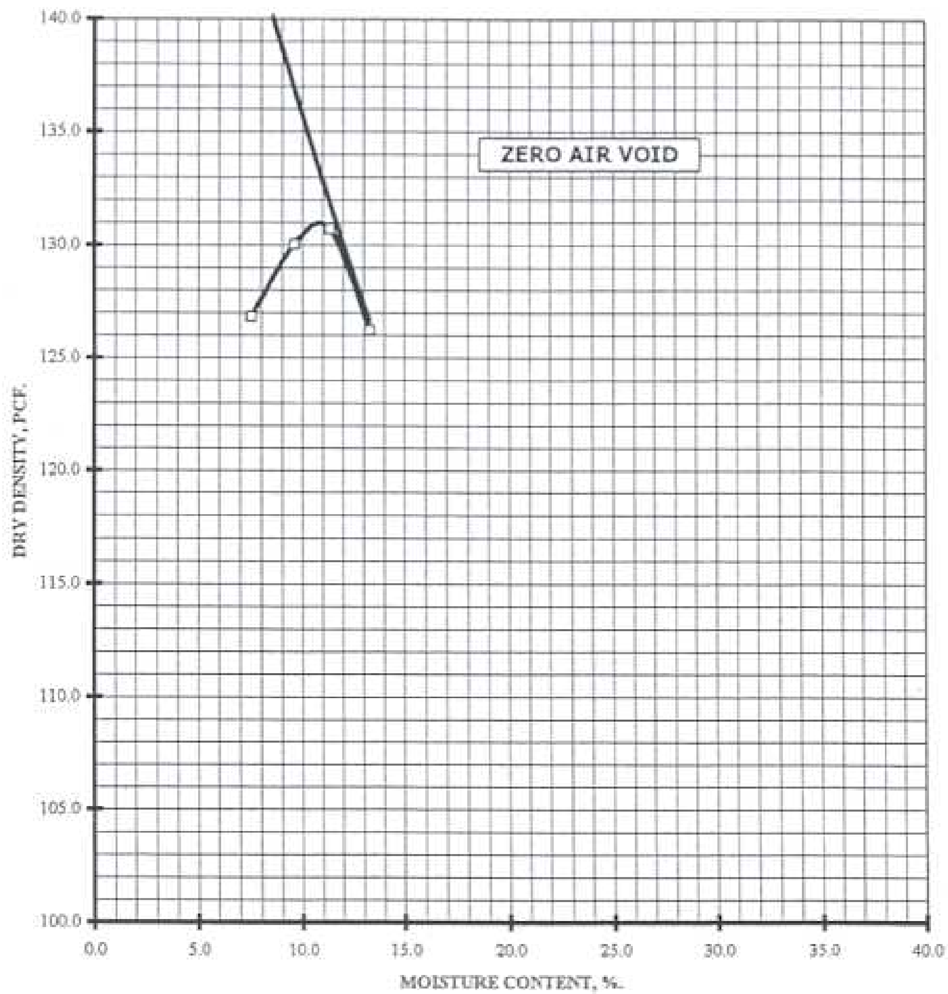
☐ ASTM D698 ■ ASTM D1557

Project No.:	1750-01	Project Name	Land Development Building	Date:	10/25/02
Sample No.:	CB 1-1	Boring/Trenc	T-1	Depth, ft.: 0.5-2.0	Tested By: JCS
Description:	Light Brown Silty Sand with Abundant Gravel			Checked By:	MLH
Sample Location:				Lab. No.:	2-553

Vol., Mold, cf.:	0.03330	No. of Layers:	5	Hammer Drop:	18
Method:	B	Blows/Layer:	25	Hammer Weight:	10

Trial Number		+2	+4	+6	+8	
Container Number		1	2	3	54	
Wet Soil + Container	(gms.)	1082.90	1074.10	1096.40	1044.70	
Dry Soil + Container	(gms.)	1018.40	993.50	1000.30	939.90	
Container Weight	(gms.)	163.50	152.30	151.50	152.20	
Weight of Water	(gms.)	64.50	80.60	96.10	104.80	
Weight of Dry Soil	(gms.)	854.90	841.20	848.80	787.70	
Moisture Content	(%)	7.5	9.6	11.3	13.3	
Wet Soil + Mold	(gms.)	4086	4178	4224	4186	
Weight of Mold	(gms.)	2026	2026	2026	2026	
Wet Weight of Soil	(lbs.)	4.54	4.74	4.85	4.76	
Wet Unit Weight	(pcf.)	136.4	142.5	145.5	143.0	
Dry Unit Weight	(pcf.)	126.8	130.0	130.7	126.2	

% Rock Retained					
3/8"sieve	3/4"sieve	Maximum Dry Density, pcf.:	131.0	Max. w/ Rock Correction	137.0
14.0	4.0	Optimum Moisture Content:	10.5	Adj. Optimum Moisture %	8.7
		Est. Specific Gravity:	2.78	Est. Specific Gravity:	2.78

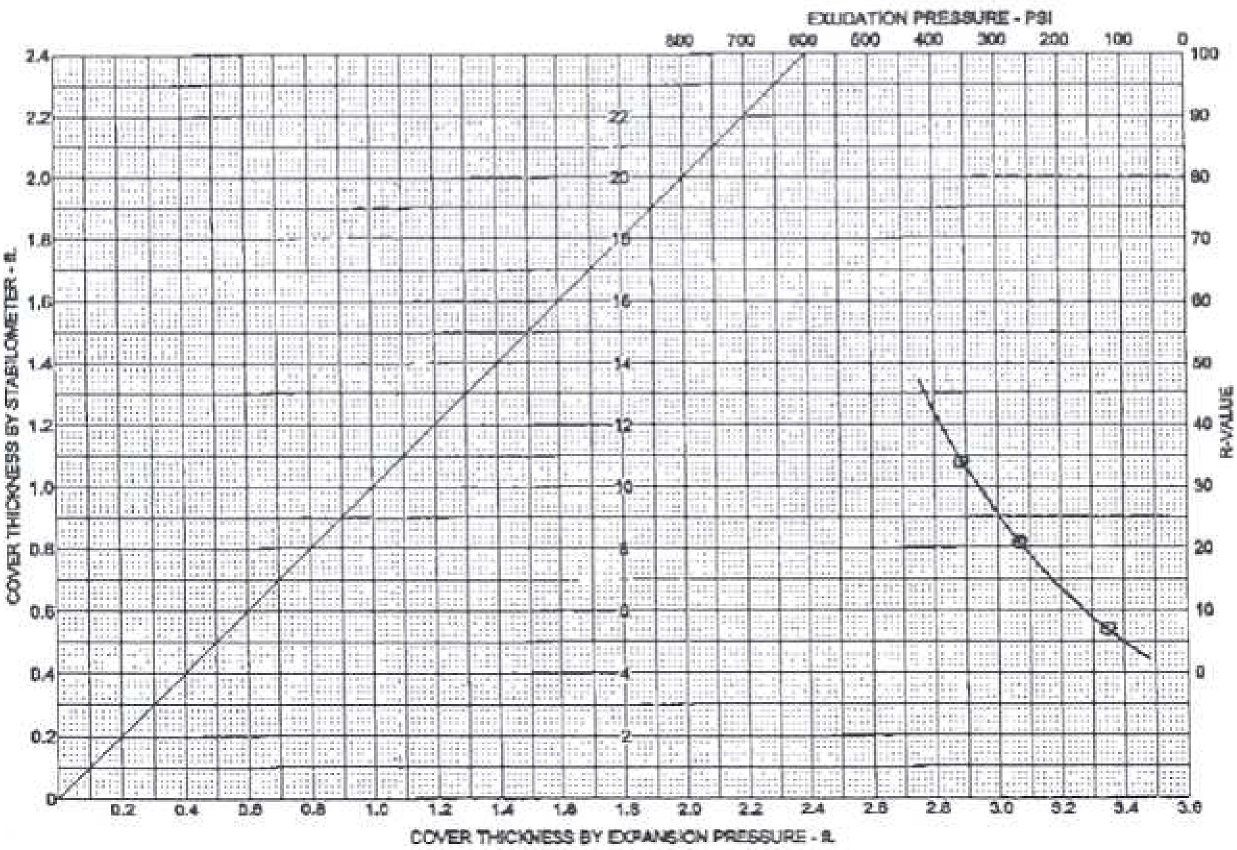


HOLDREGE & KULL

RESISTANCE VALUE

CALTRANS 301

SPECIMEN	A	B	C
EXUDATION PRESSURE, PSI	127	271	358
EXPANSION DIAL (.0001")	8	34	56
EXPANSION PRESSURE, PSF	35	147	242
RESISTANCE VALUE, R	7	21	34
% MOISTURE AT TEST	19.9	18.0	17.1
DRY DENSITY AT TEST, PCF	107	110	111
R VALUE @ 300 PSI EXUDATION PRESSURE		28	
R VALUE BY EXPANSION PRESSURE (TI=)			



WALLACE • KUHL & ASSOCIATES INC.	HOLDREGE AND KULL	WKA NO:	4848.01
	Holdrege and Kull Job Name: Land Development Center	DATE:	10/28/02
	Sample Comp-1; Reddish brown fine sandy clayey silt	LAB NO:	9808

Sunland Analytical

11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 10/30/2002
Date Submitted 10/24/2002

To: Jason Muir
Holdrege & Kull
792 Searls Ave.
Nevada City, CA 95959

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following:
Location : LAND DEVEL. 1750-01 Site ID : 2-553.
Thank you for your business.

* For future reference to this analysis please use SUN # 38301-73800.

SOIL ANALYSIS

Saturation Percent (SP)	45		Soil Texture	Loam
pH	5.47			
E.C.	0.15	mmho/cm		
Tot.Dissolved Salts	96	ppm		
Infiltration Rate (0% Slope)	0.54	in/hr		
% Organic Matter	7.1			
C.E.C.	12.8	meq/100g		
Sodium Absorption Ratio (SAR)	1.7			
Exchangable Sodium Percent (ESP)	1.2			
Lime Req.	45.9	#/1000 sq.ft.		
est. Nitrogen Release	4.1	#/1000 sq.ft.		

			Very Low / Low / Adequate / Excessive
Nitrate	1.98	ppm	*
Phosphorus	7.26	ppm	******
Potassium	60.83	ppm	**********
Sulfur	2.44	ppm	****
Chloride	No Test		
Carbonates	No Test		
Sodium	34.06	ppm	
Calcium	1898.79	ppm	*******************
Magnesium	360.25	ppm	************
Boron	0.15	ppm	****
Copper	No Test		
Iron	No Test		
Manganese	No Test		
Zinc	No Test		

Very Low | Low | Adequate | Excessive

Sunland Analytical
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

DATE 10/30/2002
SUN NUMBER 73800

Information requested by:
Jason Muir
Holdrege & Kull

Information for:
LAND DEVEL. 1750-01
Sample ID: 2-553

SOIL RECOMMENDATIONS FOR LANDSCAPE GARDENING

SOIL pH (Acidity and Alkalinity)

The pH of this sample indicates the soil is moderately acid and should be modified for non acid-tolerant plants. Apply 46 pounds of Lime per 1000 sq.ft. and work into ground before planting.

DISSOLVED SALTS (Indicated by E.C. & TDS)

These conditions are in the normal range for plant growth.

SOIL TEXTURE AND RATE OF WATER INFILTRATION

The infiltration rate for all soil textures decreases with increasing ground slope. At 0 to 4%, 5 to 8%, 9 to 12%, 13 to 16% and above 16% the infiltration rate of this sample decreases from 0.54 to 0.43, 0.32, 0.22, 0.14, respectively. Infiltration rate also decreases with percent of ground cover and by compaction.

WATER PENETRATION OF SOIL DUE TO CHEMICAL CHARACTERISTICS

When exchangable Sodium increases in the soil, water penetration decreases. Based on SAR and ESP values this sample has no penetration problem due to soil Sodium. No Gypsum required.

ORGANIC MATTER

Organic matter provides a slow nitrogen release and aids water retention. This sample has a moderate Organic Matter content.
To maintain moisture and provide sustained nitrogen release a level of 10% organic matter is recommended. Use amending material that is approximately 75% organic matter (i.e. many ground fir barks). Based on the analysis of this soil sample apply 1 yards per 1000 sq.ft. Spread evenly and blend into the top six inches of soil. It is a reasonable practice to apply a top dressing of 3 inches of organic mulches to aid water penetration and retention.

SOIL BORON

Boron concenrations are in a range allowing normal plant growth.

Sunland Analytical

11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

DATE 10/30/2002
SUN NUMBER 73800

Information requested by:
Jason Muir
Holdrege & Kull

Information for:
LAND DEVEL. 1750-01
Sample ID: 2-553

SOIL RECOMMENDATIONS FOR LANDSCAPE GARDENING

SOIL MACRONUTRIENTS : NITROGEN-PHOSPHORUS-POTASSIUM (N-P-K)
GENERAL N-P-K RECOMMENDATION

Use ONE of these NPK preparations for the first fertilizer application.

Standard NPK Fertilizer Preparations	6-20-20	5-20-10	16-16-16	0-10-10	28-3-4	21-0-0	Customer Choice None
#/1000 sq.ft.	20	23	N/A	N/A	N/A	N/A	**

GRASS OR SOD PREPARATION

Till in organic matter, N,P,K and micro nutrients in addition to any lime gypsum or sulfur as directed above. Smooth soil surface and follow seed or sod producers direction for moisture and product application.

TREES AND SHRUBS

Excavate holes for planting shrubs and trees to at least twice the volume of the container. Prepare backfill for tree and shrub planting holes by mixing three parts of native soil (or imported top soil) with one part organic amendment (preferably nitrogen and iron fortified) and 2.5 pounds of 6-20-20 per yard of mix. For extended fertilization, place slow release fertilizer tablets in each hole per manufacturer's instructions. If 6-20-20 was not directly added to backfill mix, during backfill apply uniformly 1/2 oz of 6-20-20 per gallon containers, 2.5 oz per 5 gallons, 6 oz per 24 inch boxes.

Summary and Suggested Sequence of Soil Improvements (#/1000 Sq.Ft.)

Lime	46	#
Organic Amendment	1	Yd./1000 Sq.Ft. Bulk organic amendment (nitrofied).
N-P-K Fertilizer	See above chart	
Sulfate-Sulfur	1	# Ammonium Sulfate

Maintenance Fertilization

Apply 5 pounds of Ammonium sulfate (21-0-0) per 1000 sq.ft.every month until plants become established. After established, apply 28-3-4 (or similar preparation) to provide desired growth rate and color.

GEOTECHNICAL ENGINEERING REPORT
for
AUBURN JUSTICE CENTER
Placer County Project No. 4674
Dewitt Center
Auburn, California

Prepared for:
Placer County Department of Facility Services
11476 C Avenue
Auburn, California 95603

Prepared by:
Holdrege & Kull
792 Searls Avenue
Nevada City, California 95959

Project No. 1751-01
November 4, 2002

HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS

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FACILITY SERVICES

2003 JUN -2 PM 12:52

Project No. 1751-01
May 30, 2003

Mr. Jerry Minta
Placer County Department of Facility Services
11476 C Avenue
Auburn, California 95603

Reference: ***Auburn Justice Center***
Placer County Project No. 4674
Dewitt Center
Auburn, California

Subject: ***Revision of Geotechnical Engineering Report***

Dear Mr. Minta:

Attached are revisions to Table 5.2.6.1 of our *Geotechnical Engineering Report for Auburn Justice Center* dated November 4, 2002. The original page 27 of the report should be replaced with the attached page. Several of the recommended asphalt concrete and baserock thicknesses were inadvertently transposed in the original table.

Please contact us if you have any questions.

Sincerely,

HOLDREGE & KULL

REGISTERED PROFESSIONAL ENGINEER
JASON W. MUIR
No. 60167
Exp. 6/04
CIVIL
STATE OF CALIFORNIA

Jason Muir
C.E. 60167

attachment: Table 5.2.6.1 - Alternate Equivalent Pavement Sections

copies: 3 of attachment to Jerry Minta / Placer County Department of Facility Services

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Table 5.2.6.1 - Alternate Equivalent Pavement Sections
Auburn Justice Center

Traffic Index: 4 Design R-Value: 17	Pavement Section Alternate A (feet)	Pavement Section Alternate B (feet)
Caltrans Section 26, Standard Specifications, Asphalt Concrete	0.20	---
Caltrans Section 26, Class 2 Baserock 95% compaction	0.50	---
Subgrade Soil 95% compaction	0.50	0.50
Traffic Index: 5 Design R-Value: 17	Pavement Section Alternate A (feet)	Pavement Section Alternate B (feet)
Caltrans Section 26, Standard Specifications, Asphalt Concrete	0.20	0.25
Caltrans Section 26, Class 2 Baserock 95% compaction	0.75	0.65
Subgrade Soil 95% compaction	0.50	0.50
Traffic Index: 6 Design R-Value: 17	Pavement Section Alternate A (feet)	Pavement Section Alternate B (feet)
Caltrans Section 26, Standard Specifications, Asphalt Concrete	0.25	0.30
Caltrans Section 26, Class 2 Baserock 95% compaction	0.90	0.80
Subgrade Soil 95% compaction	0.50	0.50

The upper 6 inches of native soil should be scarified and recompacted to a minimum of 95 percent of the maximum dry density per ASTM D1557 or CTM 216. The upper 12 inches of imported granular fill, if used, and all baserock must also

HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS

RECEIVED

'02 DEC -2 A9:14

Project No. 1751-01
November 26, 2002

Mr. Jerry Minta
Placer County Department of Facility Services
11476 C Avenue
Auburn, California 95603

Reference: ***Auburn Justice Center***
Placer County Project No. 4674
Dewitt Center
Auburn, California

Subject: ***Seismic Design Criteria***

Dear Mr. Minta:

Our geotechnical engineering report dated November 4, 2002 for the Auburn Justice Center referenced seismic design criteria set forth in the 1998 California Building Code. This letter confirms that the design criteria presented in our report also meet the requirements of the 2001 California Building Code.

We appreciate the opportunity to provide geotechnical engineering services for your project. Please contact us if you need any additional information.

Sincerely,

HOLDREGE & KULL

REGISTERED PROFESSIONAL ENGINEER
JASON MUIR
No. 60167
Exp. 6/04
CIVIL
STATE OF CALIFORNIA

Jason Muir
C.E. 60167

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HK HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS

Project No. 1751-01
November 4, 2002

Mr. Jerry Minta
Placer County Department of Facility Services
11476 C Avenue
Auburn, California 95603

Reference: ***Auburn Justice Center***
Placer County Project No. 4674
Dewitt Center
Auburn, California

Subject: ***Geotechnical Engineering Report***

Dear Mr. Minta:

This report presents the results of our geotechnical engineering investigation for the proposed Auburn Justice Center to be located at the Dewitt Center in Auburn, California. We understand that the project, as currently proposed, will include the construction of a roughly 50,000 to 60,000 square foot justice center building, a small enclosed parking building, and roughly 250,000 square feet of paved parking areas.

The findings and recommendations presented in this report are based on our subsurface investigation, laboratory test results, engineering analysis, and our experience with subsurface conditions in the area. Our opinion is that the project can be completed as proposed, provided the recommendations presented in this report are implemented. Our primary concerns, from a geotechnical engineering standpoint, are existing fill and stockpiled soil encountered across much of the central portion of the site and relatively shallow, resistant rock. We should be allowed to perform testing and observation services during grading to confirm our recommendations.

Please contact us if you have any questions regarding our observations or the recommendations presented in this report.

Sincerely,

HOLDREGE & KULL

Prepared by: Reviewed by:

Jason Muir
C.E. 60167

Charles R. Kull
G.E. 2359/C.E.G. 1622

copies: 3 to Jerry Minta / Placer County Department of Facility Services J:\WPDOCS\RPT\1751-01.GTK

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APPENDICES

1 INTRODUCTION

At the request of Mr. Jerry Minta of the Placer County Department of Facility Services, Holdrege & Kull (H&K) performed a geotechnical investigation at the proposed Auburn Justice Center project site in the Dewitt Center in Auburn, California. The geotechnical investigation was performed consistent with the scope of services presented in our proposal for the project dated August 27, 2002 (revised August 28, 2002), a copy of which is included as Appendix A of this report. For your review, Appendix B contains a document prepared by ASFE entitled *Important Information About Your Geotechnical Engineering Report*, which summarizes the general limitations, responsibilities, and use of geotechnical reports.

1.1 SITE DESCRIPTION

The approximately 10-acre Auburn Justice Center project site is located at the Dewitt Center in Auburn, California. The site is bordered by Richardson Drive to the east and northeast and by county facilities and developed commercial property in other directions. In general, the site slopes gently towards the south and west. Elevations range from approximately 1420 feet above mean sea level (MSL) near the southeast corner of the site to approximately 1403 feet above MSL near the central western site boundary. Soil was stockpiled across much of the central portion of the site. The depth of stockpiled soil was likely greater than 10 feet at some locations. Rock outcrop was observed near the southeast and central-east portions of the site. A southwest trending drainage bisects the southwest side of the property; the northeast portion of the drainage had been confined in a culvert and backfilled at an earlier date. In general, the site was sparsely vegetated. Dry grasses covered portions of the site, and dense blackberry bushes obscured the ground surface along the banks of the southwest trending drainage.

1.2 PROPOSED IMPROVEMENTS

Our understanding of the proposed improvements are based on our conversations with Mr. Jerry Minta of the Placer County Department of Facility Services and our review of a conceptual site plan prepared by Beverly Prior Architects (undated). The Justice Building and ancillary building are to be located centrally within the project site, immediately southwest of Richardson Drive between B Avenue and C

Avenue. A small, enclosed parking building and gravel parking area are to be constructed in the western portion of the site, north of B Avenue. Paved parking areas are to be located west of Richardson Drive and northwest of B Avenue, around the proposed Justice Building and ancillary building, and southwest of Richardson Drive and southeast of C Avenue. We understand that earthwork improvements may include up to 12 feet of cut in the southeast portion of the site and up to 12 feet of fill in the central portion of the site.

1.3 PURPOSE

We performed a geotechnical engineering investigation at the site, collected soil samples for laboratory testing, and performed engineering calculations to provide foundation and retaining wall design criteria, grading and drainage recommendations, and pavement design for the project.

1.4 SCOPE OF SERVICES

To prepare this report, we performed the following scope of services:

- We performed a site investigation, including a literature review and a limited subsurface investigation.
- We collected relatively undisturbed soil samples and bulk soil samples from selected exploratory trenches.
- We performed laboratory tests on select soil samples obtained during our subsurface investigation to determine their engineering material properties.
- Based on observations made during our subsurface investigation and the results of laboratory testing, we performed engineering calculations to provide foundation design criteria, grading and drainage recommendations, and pavement design for the project.

2 SITE INVESTIGATION

We performed a site investigation to characterize the existing site conditions and to develop geotechnical engineering recommendations and design criteria for earthwork and structural improvements. Our site investigation included a literature review and field investigation as described below.

2.1 LITERATURE REVIEW

As a part of our site investigations, we reviewed the Geologic Map of the Sacramento Quadrangle published by the California Department of Conservation, Division of Mines and Geology. The geologic map indicated that the project site is underlain by Paleozoic aged metavolcanic rock. The Paleozoic era spans the period of time between 230 and 600 million years before present (MYBP).

According to the *Soil Survey of Placer County, California, Western Part* (Soil Survey) (United States Department of Agriculture Soil Conservation Service and issued July 1980), the soil class associated with the project site is the Auburn silt loam. This soil type is described as a shallow, residually formed, undulating, well drained soil underlain by vertically tilted metamorphic rock. The typical surface layer is strong brown silt loam extending to an approximate depth of 4 inches below the ground surface (bgs). The surface layer is underlain by yellowish red silt loam. Basic schist is typically encountered at a depth of 20 inches bgs. The soil survey describes the soil as having severe limitations to building development due to the relatively shallow depth to resistant rock.

We reviewed the Fault Activity Map of California and Adjacent Areas published by the California Department of Conservation Division of Mines and Geology (CDMG) in 1994. The fault activity map indicated that the Dewitt Fault was located in the immediate vicinity of the project site. A portion of the Dewitt Fault was described by the fault activity map as having shown quaternary displacement (during the past 1.6 million years) based on geomorphic evidence. We were not able to determine the exact location of the Dewitt Fault with respect to the proposed improvements due to the scale of the fault activity map.

We also reviewed the General Geology of the Auburn 15-Minute Quadrangle, which is included in the *Mineral Land Classification of the Auburn 15' Minute Quadrangle* published by CDMG in 1984. The general geology map indicates a fault, in the apparent alignment of the Dewitt Fault, passes through the Dewitt Center east of the project site.

2.2 FIELD INVESTIGATION

We performed our field investigation on October 18 and 22, 2002. During our field investigation, we observed the local topography and general surface conditions and performed a subsurface investigation. The surface and subsurface conditions observed during our field investigation are summarized in the following sections.

Our subsurface investigation included the excavation of fifteen exploratory trenches across the project site, at the approximate locations shown on Figure 1. We excavated to depths ranging from 2.5 to 9 feet using a Case 580 backhoe equipped with a 24-inch bucket. An engineer from our firm logged the soil conditions revealed in the exploratory trenches and collected relatively undisturbed and bulk soil samples for laboratory testing.

2.2.1 Surface Conditions

In general, the site sloped gently towards the south and west. Elevations ranged from approximately 1420 feet above mean sea level (MSL) near the southeast corner of the site to approximately 1403 feet above MSL near the central western site boundary. Soil was stockpiled across much of the central portion of the site. Rock outcrop was observed near the southeast and central-east portions of the site. A southwest-trending drainage bisected the southwest side of the property; the northeast portion of the drainage had been confined in a culvert and backfilled at an earlier date. In general, the site was sparsely vegetated. Dry grasses covered portions of the site, and dense blackberry bushes obscured the ground surface along the banks of the southwest trending drainage.

2.2.2 Subsurface Soil Conditions

The soil conditions described in the following paragraphs are generalized, based on our observations of soil revealed in our fifteen exploratory trenches. More detailed information can be found in the trench logs in Appendix C.

Excluding areas of fill and stockpiled soil, our exploratory trenches revealed that near-surface soil across much of the site was red-brown to orange-brown, loose to medium dense, sandy silt with minor clay content and variable gravel content. Topsoil had been removed across much of the site as a result of previous grading.

Severely to moderately weathered, highly fractured, metamorphic rock was generally encountered at depths ranging from 1 to 2.5 feet bgs. Excavation with the Case 580 backhoe was difficult at many of the exploratory trench locations at depths generally ranging from 1.5 to 4 feet bgs in moderately weathered metavolcanic rock.

Fill encountered in trenches T-3 and T-4 ranged from 1.5 to 2.5 feet in depth and was described as brown to orange-brown, medium dense, silty sand with abundant angular rock and trace concrete and asphalt. The fill was underlain by completely to moderately weathered metamorphic rock. Trenches T-3 and T-4 were located in a proposed cut area.

Approximately 6 feet of fill was encountered in trench T-6, which was located along the alignment of a southwest-trending drainage that bisected the property. Based on our observation of surface conditions, the northeast portion of the drainage, in the vicinity of T-6, was replaced with a culvert and backfilled at an earlier date. The near surface fill was described as loose, sand silty and silty sand with angular gravel. Asphalt concrete pavement was encountered approximately 1 foot bgs. Fill underlying the pavement was described as brown, dense, silty sand with angular gravel and minor clay. Collection of relatively undisturbed samples of the fill was not feasible due to its relatively high density and rock content.

Trenches T-8 through T-15 were excavated in the vicinity of the stockpiled soil located centrally within the site. Up to 8 feet of old fill and stockpiled soil was observed in our exploratory trenches. Stockpiled soil depth is likely to be deeper than 10 feet near the tallest portion of the soil stockpile area. In general, the stockpiled soil and existing fill was described as brown to red-brown, loose, silty sand with angular rock, minor clay, and trace debris. Concrete, asphalt, wood, nylon rope and plastic were observed in minor quantities within the stockpiled soil and existing fill. A layer of brown to gray, loose, silty sand with abundant organic material and angular rock was encountered from 3 to 5 feet bgs in trench T-8. Fill was not encountered in trenches T-11 and T-14.

2.2.3 Groundwater Conditions

During our site investigation, we did not observe seepage in the sidewalls of exploratory trenches, nor did we encounter groundwater in our exploratory trenches. Our investigation was performed at the end of the dry season.

3 LABORATORY TESTING

We performed laboratory tests on selected soil samples collected from our exploratory trenches to determine their engineering properties. Collection of relatively undisturbed samples was limited in many of the exploratory trenches by the relatively shallow depth to rock. Laboratory test results were used to provide geotechnical engineering recommendations and design criteria for the proposed improvements. We performed the following laboratory tests:

- Moisture Content,
- Density (unit weight),
- Direct Shear Strength,
- Compaction Curve, and
- Resistance Value (R-Value).

Moisture/density and direct shear test results are summarized in Table 3.1 below. Graphical direct shear, compaction curve, and R-Value test results are presented in Appendix D.

Table 3.1 - Summary of Moisture/Density and Direct Shear Testing

Trench Number	Sample Number	Depth (feet)	Dry Density (pcf)	Moisture Content (%)	Shear Friction Angle (degrees)	Shear Cohesion (psf)
T-1	BT 1-1	1.0	98.3	6.1	--	--
T-14	BT 14-1	1.0	112.3	6.1	32	361
T-14	BT 14-2	1.5	104.1	9.6	--	--

Compaction curve testing for bulk soil sample CB 6-1, obtained from existing fill exposed in exploratory trench T-6, resulted in a maximum dry density of 124.0 pounds per cubic feet (pcf) and an optimum moisture content of 15.0 percent, per

ASTM D1557 guidelines. The sample was described as brown, silty sand with angular gravel.

R-value testing was performed for composite soil sample COMP-1, described as red-brown, gravelly silt with clay. Testing resulted in an R-value of 17. Sample COMP-1 was composed of portions of samples CB 2-1, CB 4-1, CB 5-1, and CB 6-1.

4 CONCLUSIONS

The following conclusions are based on our field observations, laboratory test results, and our experience in the area.

- Our opinion is that the site is suitable for the proposed improvements, provided that the geotechnical engineering recommendations and design criteria presented in this report are incorporated into the project plans.

- Prior to grading and construction, we should be allowed to review the proposed grading plan and structural improvements to confirm our recommendations.

- We observed soil/rock conditions to a maximum depth of approximately 9 feet bgs. Exploration depth was limited by resistant metamorphic rock. The soil and groundwater conditions below that depth are unknown. Much of the central portion of the improvement area was covered with existing fill and stockpiled soil that was deeper than 10 feet in some areas. In areas that were not covered with existing fill and stockpiled soil, the surface soil was generally comprised of native, residual soil underlain at shallow depth by severely to moderately weathered rock.

- Based on our observations of surface and subsurface soil/rock conditions, our primary concern, from a geotechnical standpoint, is the presence of relatively shallow, resistant rock and the existing fill and stockpiled soil across much of the central portion of the site. The existing fill and stockpiled soil was generally loose and incapable of supporting the proposed improvements. The fill and stockpiled soil observed in our exploratory trenches contained minor amounts of organic material and miscellaneous debris. From a geotechnical engineering standpoint, much of the existing fill and stockpiled sol is likely to be suitable for use as fill, provided that organic material is removed and the fill is removed, conditioned and replaced according to the recommendations contained in this report.

- During our subsurface investigation, excavation with a Case 580 backhoe was difficult at depths ranging from 1 to 4 feet below original ground surface due to the presence of moderately to slightly weathered metavolcanic rock.

- We did not observe expansive clay soil during our subsurface investigation. Based on our experience in the area, however, expansive soil may be present at the site. Potentially expansive clay, when present, typically overlies the variably weathered metamorphic rock encountered at a depth of 1 to 4 feet below original ground surface at the project site. In addition, the soil stockpiled at the site may contain zones of potentially expansive clay soil. We should be allowed to observe and test potentially expansive soil, if encountered during grading, and provide additional recommendations for mixing, use in non-structural areas, or offhaul.

- During our investigation, we did not encounter subsurface seepage in our exploratory trenches. We anticipate that seasonal subsurface seepage will be encountered near the surface soil/metamorphic rock interface, particularly during or immediately following the rainy season. In addition, our experience in the region has revealed that groundwater may be perched on rock in relatively level or gently sloping areas well into the summer months. If encountered, perched groundwater may require ripping and air drying of subgrade soil or lime treatment to facilitate grading, even during the summer months. We anticipate that wet soil conditions will be encountered in the southwest-trending drainage that bisects the site. Recommendations pertaining to shallow subsurface seepage are presented in the *Construction Dewatering* section on page 17.

- Trench T-6 was excavated in existing fill on the northeast end of the alignment of a southwest-trending drainage. The upper foot of fill was relatively loose. The deeper fill was described as relatively dense, silty sand with angular gravel and minor clay. Collection of relatively undisturbed samples of the backfill was not feasible due to the high rock content. Based on our observation of soil conditions at the location of trench T-6, the upper 12 to 18 inches of the fill would have to be reworked to achieve acceptable densities. The fill below that depth should be observed and tested, if possible, at other locations during site grading to confirm its suitability to support the proposed improvements.

- We reviewed the Fault Activity Map of California and Adjacent Areas published by the California Department of Conservation Division of Mines and Geology (CDMG) in 1994. The fault activity map indicated that the Dewitt Fault was located in the vicinity of the project site. We also reviewed the General Geology of the Auburn 15-Minute Quadrangle, which is included in the *Mineral Land Classification of the Auburn 15' Minute Quadrangle* published by CDMG in 1984. The general geology map indicates a fault, in the apparent alignment of the Dewitt Fault, passes through the Dewitt Center east of the project site.

5 RECOMMENDATIONS

The following geotechnical engineering recommendations are based on our understanding of the project as currently proposed, our field observations, the results of our laboratory testing program, engineering analysis, and our experience in the area.

5.1 GRADING

We understand that the proposed earthwork improvements may include up to 12 feet of cut and fill. We anticipate that resistant rock will be encountered at depths of 0 to 4 feet below the ground surface that may be difficult to excavate with conventional equipment. Rock at depth may be difficult to excavate with conventional grading equipment. Large equipment such as a Caterpillar D9R or D10R may be required. The effectiveness of the excavation will depend on underlying joints and fractures in the rock. Ripper teeth will have difficulty penetrating slightly weathered rock that has little or no fractures or joints. Blasting might be necessary in deeper cuts.

Although we did not observe fibrous minerals during our field investigation, we have encountered asbestiform minerals at a nearby site during a previous investigation. Therefore, asbestiform minerals may be encountered on the project site during grading. Asbestos has been recognized by the State of California Environmental Protection Agency and Air Resources Board to be a carcinogen. Grading in areas of fibrous serpentinite rock will require an asbestos mitigation plan. The plan would address air monitoring, laboratory testing, special handling and input from Placer County Division of Environmental Health and the Placer County Air Resources Board.

The following sections present our grading recommendations. The grading recommendations address site preparation for fill placement, fill construction, fill slope grading, erosion control, subsurface drainage, surface water drainage, and plan review and construction monitoring.

5.1.1 Clearing and Grubbing

Areas proposed for grading and fill placement should be cleared of vegetation, loose fill and stockpiled soil, loose surface soil, and other deleterious materials as described below. Exploratory trenches T-8 through T-15 were excavated in the vicinity of existing fill and stockpiled soil in the central portion of the site. A representative of H&K should determine the lateral and vertical extent of the existing fill and stockpiled soil during grading. Existing fill encountered below 12 to 18 inches in trench T-6 may be suitable to support the proposed improvements; however, the fill should be observed by H&K and tested, if possible, during grading to confirm its suitability.

1. Strip and remove the top 1 to 2 inches of soil containing shallow roots and other deleterious materials. Stripped soil, highly organic topsoil or soil containing shallow vegetation, roots and other deleterious materials can be stockpiled onsite and used in landscape areas, but is not suitable for use as fill.

2. Overexcavate any relatively loose debris and soil that is encountered in our exploratory trenches or any other onsite excavations to underlying, competent material.

3. Overexcavate any loose or untested, existing fill to underlying competent soil, as determined by a representative of H&K.

4. Remove all rocks greater than 6 inches in greatest dimension (oversized rock) from the top 12 inches of soil, if encountered. Oversized rock may be used in landscape areas or removed from the site.

5. Fine grained, potentially expansive soil, as determined by H&K, that is encountered during grading within proposed building locations and paved areas should be mixed with granular soil or overexcavated and stockpiled for removal from the project site or for later use in landscape areas. A typical mixing ratio for granular to expansive soil is 4 to 1. The actual mixing ratio should be determined by H&K.

6. Vegetation, deleterious materials, and oversized rocks not used in landscape areas, drainage channels, or other non-structural uses should be removed from the site.

5.1.2 Cut Slope Grading

Cut slopes, if proposed, should be graded with a maximum slope gradient of 2:1, horizontal:vertical (H:V), and should not exceed approximately 8 feet in height. Cuts slopes steeper than 2:1, H:V, would possible in the weathered rock typically encountered 1 to 4 feet below original ground surface at the project site. If cut slopes are proposed to be steeper than 2:1, H:V, and/or with a vertical height greater than 4 feet, we should be allowed to review the proposed slope configuration and provide revised recommendations, if appropriate.

5.1.3 Soil Preparation for Fill Placement

After site clearing, the exposed surface soil should be prepared for placement of compacted fill as described below.

1. The surface soil should be scarified to a minimum depth of 8 inches below the existing ground surface and then uniformly moisture conditioned to within approximately 2 percentage points of the ASTM D1557 optimum moisture content.

2. The scarified soil should then be compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density. The

moisture content, density and relative percent compaction should be verified by our construction quality assurance (CQA) monitor. The earthwork contractor should assist our CQA monitor by excavating test pads with onsite earth moving equipment.

3. Construction quality assurance tests should be performed using the following minimum testing frequencies, or as determined by the project geotechnical engineer.

Table 5.1.3.1 - Minimum Testing Frequencies for Native Soil Preparation

ASTM No.	Description	Test Frequency
D1557	Modified Proctor Curve	1 per 100,000 sf (1) or material change(2)
D2922	Nuclear Moisture	1 per 10,000 sf
D3017	Nuclear Density	1 per 10,000 sf

Notes: (1) sf = square feet
(2) higher testing frequency shall govern

5.1.4 Fill Placement

Fill placement should incorporate the following recommendations:

1. Soil used for fill construction should consist of uncontaminated, predominantly granular, non-expansive native soil or approved import soil. If encountered, rock used in fill should be broken into pieces no larger than 6 inches in diameter. Rocks larger than 6 inches are considered oversized material and should be stockpiled for offhaul or later use in landscape areas.

2. Proposed import soil should be predominantly granular, non-expansive and free of deleterious material. Import material that is proposed for use onsite should be submitted to H&K for approval and possible laboratory testing at least 72 hours prior to transport to the site.

3. Cohesive, predominantly fine grained, or potentially expansive soil encountered during grading should be stockpiled for removal, mixed as directed by H&K, or used in landscape areas. We observed highly expansive clay at a depth of 2 to 3 feet bgs in exploratory trenches T-3 and T-4.

4. Soil used to construct fill should be uniformly moisture conditioned to within approximately 2 percentage points of the ASTM D1557 optimum moisture content. Wet soil may need to be air dried or mixed with drier material to facilitate placement and compaction, particularly during or following the wet season.

5. Fill should be constructed by placing uniformly moisture conditioned soil in maximum 8-inch-thick loose, horizontal lifts (layers) prior to compacting.

6. All fill should be compacted to a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density. The upper 8 inches of fill in paved areas should be compacted to a minimum of 95 percent relative compaction.

7. Construction quality assurance tests should be performed using the following minimum testing frequencies, or as determined by the project geotechnical engineer:

Table 5.1.4.1 - Minimum Testing Frequencies for Fill Placement

ASTM No.	Description	Test Frequency
D1557	Modified Proctor Curve	1 per 3,000 cy (1) or material change(2)
D2922	Nuclear Moisture	1 per 100 cy(3)
D3017	Nuclear Density	1 per 100 cy(3)

Notes: (1) cy = cubic yards
(2) higher testing frequency shall govern
(3) A minimum of 1 test should be taken per every 18 inches of elevation change as fill is placed. Irregular fill or fill of inconsistent quality may require more frequent testing.

The moisture content, density and relative percent compaction of all fill should be verified by our CQA monitor during construction. The earthwork contractor should assist our CQA monitor by excavating test pads with the onsite earth moving equipment.

5.1.5 Differential Fill Depth

The recommendations presented in this section are intended to reduce the magnitude of differential settlement-induced structural distress associated with variable fill depth beneath structures:

1. Site grading should be performed so that cut-fill transition lines do not occur directly beneath any structures. The cut portion of the cut-fill building pads, if proposed, should be scarified to a minimum depth of 12 inches and recompacted to 95 percent relative compaction.

2. Differential fill depths beneath structures should not exceed 5 feet. For example, if the maximum fill depth is 8 feet beneath the proposed building footprint, the minimum fill depth beneath that pad should not be less than 3 feet. If a cut-fill building pad is used in this example, the cut portion would need to be overexcavated 3 feet and rebuilt with compacted fill.

5.1.6 Fill Slope Grading

Fill slopes, if proposed should be graded as described below.

1. In general, fill slopes should be no steeper than 2:1, H:V. Proposed fill slope configurations greater than approximately 8 feet in height should be reviewed by H&K. Compaction and fill slope grading must be verified by H&K in the field.

2. Fill should be placed in horizontal lifts to the grades shown on the project plans. Fill slopes should be constructed by overbuilding the slope face and then cutting it back to the design slope gradient. Fill slopes should not be constructed or extended horizontally by placing soil on an existing slope face and/or compacted by track walking.

5.1.7 Erosion Controls

Graded portions of the site should be seeded as soon as possible following grading to allow vegetation to become established prior to and during the rainy season. The following erosion controls should be installed on all cut and fill slopes, if created during grading, to reduce erosion:

1. All slopes created during grading should be hydroseeded or hand seeded/strawed with an appropriate seed mixture compatible with the soil and climate conditions of the site as recommended by the local Resource Conservation District.

2. Following seeding, jute netting should be placed and secured over the slopes to keep seeds and straw from being washed or blown away. Tackifiers or binding agents may be used in lieu of jute netting.

3. Surface water drainage ditches should be established at the top of all slopes to intercept and redirect surface water away from the slope face. Under no circumstances should surface water be allowed to run over slope faces. The intercepted water should be discharged into natural drainage courses or into other collection and disposal structures.

5.1.8 Underground Utility Trenches

Underground utility trenches should be excavated and backfilled as described below.

1. We anticipate that the contractor will encounter resistant, moderately to slightly weathered rock in excavations as shallow as 1 foot below the existing ground surface in some portions of the site. During our investigation, difficult excavation with the Case 580K backhoe was encountered at depths ranging from 1 to 4 feet below original ground surface in moderately weathered metavolcanic rock, and rock outcrop was observed at locations shown approximately on Figure 1. In addition, groundwater seepage should be anticipated in excavations which expose the soil/rock interface.

2. The California Occupational Safety and Health Administration (OSHA) requires all utility trenches deeper than 4 feet bgs to be shored with bracing equipment prior to being entered by any individuals, whether or not they are associated with the project.

3. Utilities should be placed as shallow as possible to reduce the need for blasting, pre-ripping or jack hammering of trenches.

4. We anticipate that shallow subsurface seepage may be encountered, particularly if utility trenches are excavated during the winter, spring, or early summer. The earthwork contractor may need to employ dewatering methods as discussed in the *Construction Dewatering* section on page 17 in order to excavate, place and compact the utility trench backfill materials.

5. Soil used as trench backfill should be non-expansive and should not contain rocks greater than 3 inches in greatest dimension.

6. Soil used to backfill trenches should be uniformly moisture conditioned to within approximately 2 percentage points of the ASTM D1557 optimum moisture content.

7. Trench backfill should be constructed by placing uniformly moisture conditioned soil in maximum 8-inch-thick loose lifts (layers) prior to compacting.

8. Trench backfill placed beneath the utilities (bedding) should be compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density.

9. Trench backfill soil should be compacted to a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density.

10. Trench backfill soil placed within 1 foot of the finished subgrade in road and parking lot areas should be compacted to a minimum relative compaction of 95 percent of the ASTM D1557 maximum dry density.

11. Construction quality assurance tests should be performed during utility trench backfill placement using the following minimum testing frequencies, or as determined by the project geotechnical engineer:

Table 5.1.8.1 - Minimum Testing Frequencies for Trench Backfill		
ASTM No.	Description	Test Frequency
D1557	Modified Proctor Curve	1 per 1,000 cy (1) or material change(2)
D2922	Nuclear Moisture	1 per 100 ft trench and 18 inches fill depth(2)
D3017	Nuclear Density	1 per 100 ft trench and 18 inches fill depth(2)

Notes: (1) cy = cubic yards
(2) higher testing frequency shall govern

12. The loose lift thickness, moisture, density and relative compaction of the trench backfill soil should be verified by our CQA Monitor. The earthwork contractor should assist our CQA monitor during construction by excavating test pits in the compacted trench backfill material.

5.1.9 Construction Dewatering

The earthwork contractor should be prepared to dewater excavations if seepage is encountered during grading. Seepage may be encountered if grading is performed during and immediately after the rainy season. In addition, perched groundwater may be encountered on the underlying, resistant metamorphic rock in flat to gently sloping areas even during the summer months. The following recommendations are preliminary and are not based on a groundwater flow analysis. A detailed dewatering analysis was not a part of our proposed scope of services.

1. We anticipate that dewatering of utility trenches can be performed by constructing sumps to depths below the trench bottom and removing the water with sump pumps. Additional sump excavations and pumps should be added as necessary to keep the base of excavations free of standing water when placing and compacting the trench backfill. Because of the relatively level nature of the site, the contractor should not rely on gravity alone to dewater excavations.

2. If seepage is encountered during trench excavation, it may be necessary to remove underlying saturated soil and replace it with free draining, granular drain rock enveloped in geotextile fabric. Native backfill soil can again be

used after placing the granular rock to an elevation that is higher than the encountered groundwater.

5.1.10 Subsurface Drainage

Moist or saturated soil conditions will likely be encountered, which limit grading to the drier, summer months. If subsurface seepage or groundwater conditions are encountered which prevent or restrict fill placement, subdrains may be necessary, particularly if grading is performed during or immediately following the wet season. If groundwater or saturated soil conditions are encountered during grading, we should be allowed to observe the conditions and provide site specific subsurface drainage recommendations.

5.1.11 Surface Water Drainage

Proper surface water drainage is important to the successful development of the project. We recommend the following measures to help mitigate surface water drainage problems:

1. Slope final grade in structural areas so that surface water drains away from buildings at a minimum 2 percent slope for a minimum distance of 15 feet.

2. Compact and slope all soil placed adjacent to building foundations such that water is not allowed to pond or infiltrate. Backfill should be free of deleterious material.

3. Direct downspouts to positive drainage or a closed collector pipe which discharges flow to positive drainage.

4. Construct V-ditches at the top of all cut and fill slopes to reduce surface water flow over slope faces. Typically, V-ditches should be 3 feet wide and at least 6 inches deep. Surface water collected in V-ditches should be directed away and downslope from proposed and existing building pads and driveways into a drainage channel.

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5.1.12 Grading Plan Review and Construction Monitoring

Construction quality assurance includes review of plans and specifications and performing construction monitoring as described below.

1. We should be allowed to review the final earthwork grading plans prior to construction to confirm our understanding of the project at the time of our investigation, to determine whether our recommendations have been implemented, and to provide additional and/or modified recommendations, if necessary.

2. We should be allowed to perform construction quality assurance and quality control (CQA/QC) monitoring of all earthwork grading performed by the contractor to determine whether our recommendations have been implemented, and if necessary, provide additional and/or modified recommendations.

5.2 STRUCTURAL IMPROVEMENT DESIGN CRITERIA

5.2.1 Foundations

The following foundation recommendations address foundation construction in competent native soil or fill placed, compacted, and tested in accordance with the recommendations presented in this report.

1. All footings for single story structures should be a minimum of 12 inches wide and trenched through any loose surface material and a minimum of 12 inches into competent native soil or compacted fill placed and tested in accordance with the recommendations presented in this report. Footings for two-story structures should be a minimum of 15 inches wide and trenched through any loose surface material and a minimum of 18 inches into competent native soil or compacted fill placed and tested in accordance with the recommendations presented in this report.

2. If fine grained, potentially expansive soil is encountered at the base of footings, the footing should be deepened through the clay lens into

underlying granular soil or weathered rock, as determined in the field by H&K.

3. Footing trenches should be cleaned of all loose soil and construction debris prior to placing concrete. A representative from H&K should observe the footing excavations prior to reinforcing steel and concrete placement.

4. The project structural engineer should design the footings. Minimum steel reinforcement in continuous footings should consist of two No. 4 rebar, one near the top of the footing and one near the bottom. A minimum of 3 inches of concrete coverage should surround the bars.

5. All footings with a minimum embedment depth of 12 inches in competent soil may be sized for an allowable bearing capacity of 2,500 psf for dead plus live loads. This value can be increased by 400 psf for each additional foot of embedment, up to a limiting value of 3,700 psf. Allowable bearing values may be increased by 33 percent for additional transient loading such as wind or seismic. Allowable values may be increased where rock is encountered, as determined by H&K.

6. Lateral footing resistance derived from passive earth pressure can be modeled as a triangular pressure distribution ranging from 0 psf at the ground surface to a maximum of 300d psf, where d equals the depth of the footing, in feet.

7. As an alternative to passive resistance, a coefficient of friction of 0.40 between the base of concrete footings and the soil may be used to calculate lateral resistance. Passive pressure and frictional resistance should not be combined when estimating lateral resistance. However, either approach may be considered as an additional factor of safety.

8. Footing excavations should be saturated prior to placing concrete to reduce the risk of problems caused by wicking of moisture from curing concrete.

9. A coefficient of friction for uplift of 150 psf may be used. This value should only be used for short term (wind) loading. Skin friction should be neglected within one foot of the ground surface.

10. Footing excavations should be saturated prior to placing concrete to reduce the risk of problems caused by wicking of moisture from curing concrete.

11. We anticipate that resistant rock may be encountered which limits footing trench excavation. Where footings are proposed to be constructed on competent rock, a higher allowable bearing capacity may be employed as determined by the project geotechnical engineer. Rock anchors are discussed in the following section.

5.2.2 Rock Anchors

Rock anchors or doweling may be used to provide lateral and uplift resistance where shallow, competent rock limits footing excavation. Rock anchors should only be installed in competent rock, to be determined in the field by a representative of H&K. The design of rock anchors should include the following criteria.

1. Pull-out resistance for rock anchors will generally be limited by the shear resistance between the grout and the native rock. For design purposes, a pull-out resistance of 50 pounds per square inch of grout/competent rock contact may be used. Because of the strain in the anchor steel during pull-out, we recommend that the upper 6 inches of grout/competent rock contact be neglected when sizing for uplift.

2. We recommend that the drilled hole have a minimum ½-inch annular clearance between the steel and surrounding rock. Thus, grouting a No. 4 rebar would require a 1½-inch diameter hole.

3. Lateral shear resistance for rock anchors should be designed using V_s=0.45 F_y, where F_y equals the tensile strength of the steel. To develop this shear resistance, a minimum steel embedment of 8 inches into undisturbed, competent rock should be used.

4. The anchor holes should be thoroughly cleaned with compressed air prior to grouting steel.

5. We recommend using a cement grout that has a water/cement ratio of less than 0.6 to construct rock anchors. If high strength epoxy or other adhesives are proposed, H&K should review the proposed rock anchor detail prior to construction.

6. If rock anchors are used on more than 10 percent of the foundation system of any given structure, a representative of H&K should perform pull tests on select anchors.

5.2.3 Seismic Design Criteria

The site is located in Seismic Zone 3 of the 1998 California Building Code (CBC) Seismic Zone Map. CBC seismic design coefficients are listed in Table 5.2.3.1 below.

Table 5.2.3.1 - CBC Seismic Design Coefficients

Seismic Zone Factor, Z(1)	Soil Profile Type(2)	Seismic Coefficient C_a(3)	Seismic Coefficient C_v(4)
0.30	S_B	0.30	0.30

Notes: (1) Table 16-I, 1998 CBC
(2) Table 16-J, 1998 CBC
(3) Table 16-Q, 1998 CBC
(4) Table 16-R, 1998 CBC

Our opinion is that the site may experience moderate ground shaking caused by earthquakes occurring along offsite faults. Earthquakes may cause cracking of concrete slabs, building walls, and pavement at the site.

5.2.4 Slab-on-Grade Floor Systems

A concrete slab-on-grade floor may be used in conjunction with the perimeter concrete foundation. We make the following recommendations regarding the slab-on-grade construction on competent, prepared native soil or compacted fill placed and tested in accordance with the recommendations presented in this report:

1. Slabs-on-grade should be a minimum of 4 inches thick. If floor loads higher than 250 psf, vehicle loads, or intermittent live loads are anticipated, a

structural engineer should determine the slab thickness and steel reinforcing schedule.

2. As a minimum, No. 3 rebar on 24-inch centers or flat sheets of 6x6, W2.9 x W2.9 welded wire mesh (WWM) should be used as slab reinforcement. We do not recommend using rolls of WWM because vertically centered placement of rolled mesh within the slab is difficult to achieve. All rebar and sheets of WWM should be placed in the center of the slab and supported on concrete "dobies". We do not recommend "hooking and pulling" of steel during concrete placement.

3. Slabs should be underlain by 4 inches of crushed, washed rock. The rock should be uniformly graded so that 100% passes the 1-inch sieve, with 0% to 5% passing the No. 4 sieve. The rock should be overlain by a vapor barrier at least 10 mils thick. A minimum of 2 inches of clean sand should be spread over the vapor barrier. The sand will act as a leveling pad and aid in curing the concrete. Prior to pouring concrete, the sand leveling pad should be moistened to reduce moisture withdrawal of the concrete during curing.

   The vapor barrier and sand may be omitted in areas that do not have moisture sensitive floor coverings (i.e., garage slabs and parking areas).

4. Regardless of the type of vapor barrier used, moisture can wick up through a concrete slab. Excessive moisture transmission through a slab can cause adhesion loss, warping and peeling of resilient floor coverings, deterioration of adhesive, seam separation, formation of air pockets, mineral deposition beneath flooring, odor and fungi growth. Slabs can be tested for water transmissivity in areas that are moisture sensitive. To further reduce the chance of excessive moisture transmission, a waterproofing consultant can be contacted.

5. Expansion joints should be provided between the slab and perimeter footings and bisect the length and width of the slab at intervals specified by the American Concrete Institute (ACI) or Portland Concrete Association (PCA).

6. Exterior slabs-on-grade such as sidewalks may be placed directly on compacted fill without the use of a baserock section. For exterior slabs, the native soil should be ripped to a depth of 8 inches, moisture conditioned and recompacted. To reduce the likelihood of vertical movement, exterior slabs should not be constructed on potentially expansive soil.

7. Soil should be moisture conditioned prior to placing concrete. If the soil is not moisture conditioned prior to placing concrete, moisture will be wicked out of the concrete, possibly causing shrinkage cracks. Additionally, our opinion is that the moisture conditioning the soil prior to placing concrete will reduce the likelihood of soil swell or heave following construction.

8. All deleterious material must be removed prior to placing slab concrete.

9. Exposed concrete slabs should be moisture cured for at least seven days after placement.

10. Concrete slabs impart a relatively small load on the subgrade (approximately 50 psf). Therefore, some vertical movement should be anticipated from possible expansion or differential loading. In areas where vertical movement must be minimized, the subgrade soil should be tested for swell potential. Potentially expansive soil encountered at proposed slab locations should be overexcavated and replaced with predominantly granular, non-expansive compacted fill.

5.2.5 Retaining Wall Design Criteria

The following active and passive pressures are for retaining walls in cut native soil or backfilled with granular onsite soil. If import soil is used, a representative from our firm should be allowed to observe and test the soil to determine its strength properties. The pressures exerted against retaining walls may be assumed to be equal to a fluid of equivalent unit weight.

Table 5.2.5.1 presents equivalent fluid unit weights for cut native soil and onsite, predominantly granular fill compacted per the grading recommendations presented in this report. We assume that the retained surface will be no steeper than 5% for a minimum distance of the wall height from the back of the retaining wall. If surcharge loads (such as adjacent building foundations) or live loads will be applied within a distance of the wall height from the back of the wall, we should be allowed to review the loading conditions and revise our recommendations, if necessary.

Transition from an at-rest soil pressure condition to an active soil pressure condition behind a retaining wall generally requires the wall to yield slightly. Literature suggests that this yielding may result in lateral movement of the top of the wall of up to approximately two percent of the wall height. Therefore, some wall movement should be expected during and shortly after construction. If that amount of wall movement is not acceptable, we recommend that the wall be designed using higher equivalent fluid pressures, such as the at-rest fluid pressure.

Table 5.2.5.1 - Equivalent Fluid Unit Weights (1)

Loading Condition	Retained Cut or Compacted Fill (see grading recommendations)
Active Pressure (pcf)	30
Passive Pressure (pcf)	300
At-Rest Pressure (pcf)	45
Coefficient of Friction	0.40

Note: (1) The equivalent fluid unit weights presented are ultimate values and do not include a factor of safety. The passive pressures provided assume footings are founded in competent native soil or engineered fill.

Recommendations for design and construction of retaining walls are listed below:

1. Compaction equipment should not be used directly adjacent to retaining walls unless the wall is designed or braced to resist the additional lateral pressures.

2. If any surface loads are closer to the top of the retaining wall than the height of the wall, H&K should review the loads and loading configuration. We should be allowed to review wall details and plans for any wall over 10 feet in height.

3. All retaining walls must be well drained to reduce hydrostatic pressures. Walls should be provided with a drainage blanket to reduce additional lateral forces

and minimize saturation of the backfill soil. Drainage blankets may consist of graded rock drains or geosynthetic blankets.

4. Rock drains should consist of a minimum 12-inch wide, Caltrans Class II, permeable drainage blanket, placed directly behind the wall; or crushed washed rock enveloped in a non-woven geotextile filter fabric such as Amoco 4546™ or equivalent. Drains should have a minimum 4-inch diameter, perforated, schedule 40, PVC pipe placed at the base of the wall, inside the drainrock, with the perforations placed down. The PVC pipe should be sloped so that water is directed away from the wall by gravity. A geosynthetic drainage blanket such as Enkadrain™ or equivalent may be substituted for the rock drain, provided the collected water is channeled away from the wall. If a geosynthetic blanket is used, backfill must be compacted carefully so that equipment or soil does not tear or crush the drainage blanket.

5. Drainage behind retaining walls is critical for finished interior space below grade. Drains should extend completely around below-grade portions of the exterior of the building and drain by gravity to an appropriate outfall location downslope of the building footprint. The exterior face of retaining walls should be sealed with an appropriate waterproofing sealant prior to construction of subsurface drains. To further reduce the chance of seepage and moisture transmission into finished interior space constructed below grade, a waterproofing specialist should be contacted.

5.2.6 Pavement Design

Our R-value (ASTM D301) test results of a composite soil sample collected from exploratory trenches/borings T-2, T-4, and T-5 indicated that the soil had an R-value of 17 by exudation pressure. The test results did not indicate that the soil exhibited significant expansion pressure. Recommended pavement sections for TIs of 4, 5 and 6 are presented in the following table. Compaction requirements are based on compaction relative to the maximum dry density per ASTM D 1557, Modified Proctor.

Table 5.2.6.1 - Alternate Equivalent Pavement Sections
Placer County Land Development Building

Traffic Index: 4 Design R-Value: 17	Pavement Section Alternate A (feet)	Pavement Section Alternate B (feet)
Caltrans Section 26, Standard Specifications, Asphalt Concrete	0.20	0.50
Caltrans Section 26, Class 2 Baserock 95% compaction	0.20	0.40
Subgrade Soil 95% compaction	0.50	0.50
Traffic Index: 5 Design R-Value: 17	**Pavement Section Alternate A (feet)**	**Pavement Section Alternate B (feet)**
Caltrans Section 26, Standard Specifications, Asphalt Concrete	0.20	0.75
Caltrans Section 26, Class 2 Baserock 95% compaction	0.25	0.65
Subgrade Soil 95% compaction	0.50	0.50
Traffic Index: 6 Design R-Value: 17	**Pavement Section Alternate A (feet)**	**Pavement Section Alternate B (feet)**
Caltrans Section 26, Standard Specifications, Asphalt Concrete	0.25	0.90
Caltrans Section 26, Class 2 Baserock 95% compaction	0.30	0.80
Subgrade Soil 95% compaction	0.50	0.50

The upper 6 inches of native soil should be scarified and recompacted to a minimum of 95 percent of the maximum dry density per ASTM D1557 or CTM 216. The upper 12 inches of imported granular fill, if used, and all baserock must also be compacted to a minimum of 95 percent. Subgrade and baserock density must be tested by a representative of H&K. Subgrade must be proof rolled under the observation of a representative of H&K prior to baserock placement.

Steel reinforced concrete slabs should be considered for use in loading bays, service docks, garbage facilities, or other areas where frequent, heavy vehicle loads are anticipated. The project structural engineer should determine slab thickness and steel reinforcement.

6 LIMITATIONS

The following limitations apply to the findings, conclusions and recommendations presented in this report:

1. Our professional services were performed consistent with the generally accepted geotechnical engineering principles and practices employed in northern California. This warranty is in lieu of all other warranties, either expressed or implied.

2. These services were performed consistent with our agreement with our client. We are not responsible for the impacts of any changes in environmental standards, practices, or regulations subsequent to performance of our services. We do not warrant the accuracy of information supplied by others, or the use of segregated portions of this report. This report is solely for the use of our client unless noted otherwise. Any reliance on this report by a third party is at the party's sole risk.

3. If changes are made to the nature or design of the project as described in this report, then the conclusions and recommendations presented in this report should be considered invalid by all parties. Only our firm can determine the validity of the conclusions and recommendations presented in this report. Therefore, we should be allowed to review all project changes and prepare written responses with regards to their impacts on our conclusions and recommendations. However, we may require additional fieldwork and laboratory testing to develop any modifications to our recommendations. Costs to review project changes and perform additional fieldwork and laboratory testing necessary to modify our recommendations is beyond the scope of services presented in this report. Any additional work will be performed only

after receipt of an approved scope of services, budget, and written authorization to proceed.

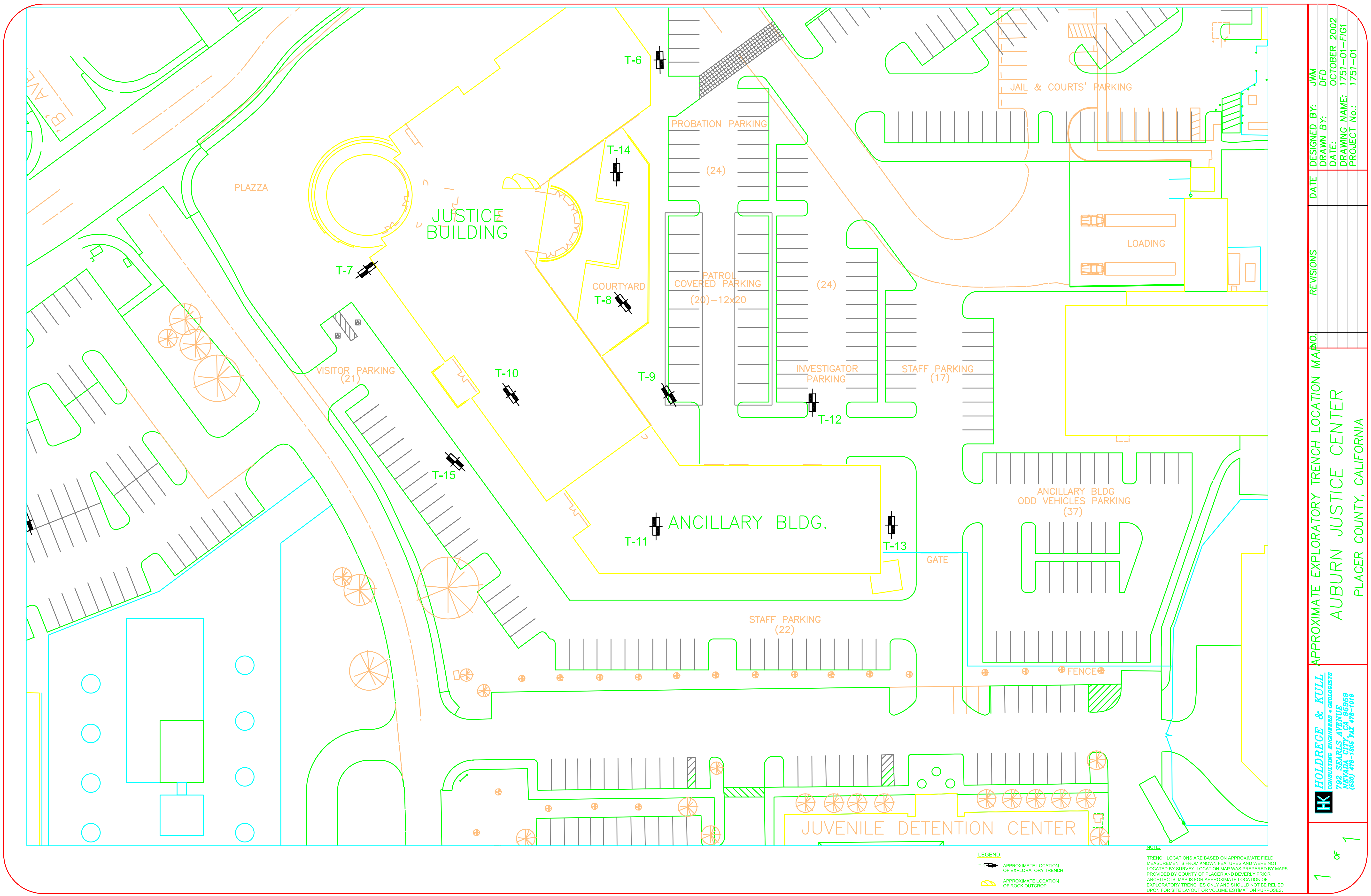
4. The analyses, conclusions and recommendations presented in this report are based on site conditions as they existed at the time we performed our surface and subsurface field investigations. We have assumed that the subsurface soil and groundwater conditions encountered at the location of our exploratory trenches are generally representative of the subsurface conditions throughout the entire project site. However, the actual subsurface conditions at locations between and beyond our exploratory trenches may differ. Therefore, if the subsurface conditions encountered during construction are different than those described in this report, then we should be notified immediately so that we can review these differences and, if necessary, modify our recommendations.

5. The elevation or depth to the groundwater table underlying the project site may differ with time and location.

6. The project site map shows approximate exploratory trench locations as determined by pacing distances from identifiable site features. Therefore, the trench locations should not be relied upon as being exact nor located with surveying methods.

7. Our geotechnical investigation scope of services did not include evaluating the project site for the presence of hazardous materials. Although we did not observe the presence of hazardous materials at the time of our field investigation, all project personnel should be careful and take the necessary precautions should hazardous materials be encountered during construction.

8. The findings of this report are valid as of the present date. However, changes in the conditions of the property can occur with the passage of time. The changes may be due to natural processes or to the works of man, on the project site or adjacent properties. In addition, changes in applicable or appropriate standards can occur, whether they result from legislation or the broadening of knowledge. Therefore, the recommendations presented in this report should not be relied upon after a period of two years from the issue date without our review.



T-6
T-14
T-7
T-8
T-9
T-10
T-11
T-12
T-13
T-15
PLAZA
JUSTICE BUILDING
COURTYARD
PROBATION PARKING
(24)
PATROL COVERED PARKING
(20)–12x20
(24)
INVESTIGATOR PARKING
STAFF PARKING
(17)
JAIL & COURTS' PARKING
LOADING
VISITOR PARKING
(21)
ANCILLARY BLDG.
GATE
ANCILLARY BLDG ODD VEHICLES PARKING
(37)
STAFF PARKING
(22)
FENCE
JUVENILE DETENTION CENTER
LEGEND
T-1
APPROXIMATE LOCATION OF EXPLORATORY TRENCH
APPROXIMATE LOCATION OF ROCK OUTCROP
NOTE:
TRENCH LOCATIONS ARE BASED ON APPROXIMATE FIELD MEASUREMENTS FROM KNOWN FEATURES AND WERE NOT LOCATED BY SURVEY. LOCATION MAP WAS PREPARED BY MAPS PROVIDED BY COUNTY OF PLACER AND BEVERLY PRIOR ARCHITECTS. MAP IS FOR APPROXIMATE LOCATION OF EXPLORATORY TRENCHES ONLY AND SHOULD NOT BE RELIED UPON FOR SITE LAYOUT OR VOLUME ESTIMATION PURPOSES.
DESIGNED BY: JWM
DRAWN BY: DFD
DATE: OCTOBER 2002
DRAWING NAME: 1751-01-FIG1
PROJECT No.: 1751-01
DATE
REVISIONS
NO.
APPROXIMATE EXPLORATORY TRENCH LOCATION MAP
AUBURN JUSTICE CENTER
PLACER COUNTY, CALIFORNIA
HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS
792 SEARLS AVENUE
NEVADA CITY, CA 95959
(530) 478-1305 FAX 478-1019
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OF
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FIGURES

APPENDIX A PROPOSAL

HOLDREGE & KULL

CONSULTING ENGINEERS • GEOLOGISTS

August 27, 2002
(Revised August 28, 2002)

FILE COPY

MAILED
8-28-02

Mr. Jerry Minta
Placer County Department of Facility Services
11476 C Avenue
Auburn, California 95603

Reference: ***Placer County Auburn Justice Center, Project No. 4674***
Dewitt Center, Auburn, California

Subject: ***Revised Proposal for Geotechnical Engineering Investigation***

Dear Mr. Minta,

We appreciate the opportunity to present this proposal to perform a geotechnical investigation for the proposed Placer County Auburn Justice Center in Auburn, California. To prepare this proposal, we reviewed a request for proposal prepared by your office and dated August 22, 2002. Our proposal was revised based on our telephone conversation with you on August 28, 2002.

We understand that the purpose of our geotechnical investigation will be to provide recommendations for site grading, foundation and retaining wall design criteria and seismic design coefficients for the proposed project. Our geotechnical investigation will specifically consider existing soil/rock conditions onsite as they relate to the construction of the proposed improvements.

As requested, we have included in our proposal an alternate scope of services that includes geotechnical engineering investigations and reports for both the Placer County Auburn Justice Center and the proposed Placer County Land Development Building (Project No. 4630). A separate geotechnical engineering report would be prepared for each project.

SCOPE OF SERVICES

Geotechnical Engineering Investigation and Report

Based on our understanding of the project, we propose the following scope of services:

1. We will review selected geologic maps and soil survey information pertaining to the project site.

2. We will perform a preliminary site visit to mark our proposed exploratory trench locations for location of existing underground utilities to be performed by others.

2. We will perform a subsurface investigation consisting of 6 to 8 exploratory trenches to maximum depths of 10 feet across the project site. During our investigation, we will obtain relatively undisturbed and bulk soil samples from our exploratory trenches for laboratory testing. An engineer or geologist from Holdrege & Kull (H&K) will log the trenches in the field.

3. We will perform laboratory tests on select soil samples. Tests will include direct shear, moisture-density and R-Value. If potentially expansive soil is encountered during our investigation within the area of proposed improvements, we will perform Atterberg Limits determination and/or expansion index testing using select soil samples. If significant fill is encountered during our investigation, we will perform compaction curve testing. Results of compaction curve testing will be used to estimate the relative density of the existing fill.

4. Using laboratory test results, we will perform the necessary calculations to provide grading recommendations, foundation and retaining wall design criteria, and seismic design coefficients.

5. Following completion of the above tasks, we will issue five copies of a geotechnical engineering report which will include:

    a. Project description;

    b. Purpose and scope;

    c. Summary of field investigation and laboratory test results;

d. Logs of exploratory trenches;

e. Description of surface and subsurface conditions encountered during our investigation;

f. Summary of our review of site geologic and soil survey information;

g. Site plan showing approximate locations of our trenches and pertinent geologic features observed during our investigation;

h. Conclusions addressing the feasibility of the project from a geotechnical engineering standpoint;

i. General recommendations for site grading and development, including site preparation and grading, fill placement, allowable cut and fill slope gradients, surface and subsurface drainage, utility trench excavation and backfill and erosion control measures;

j. Foundation design criteria, including allowable bearing capacities, coefficients of resistance to lateral forces, and seismic design coefficients;

k. Retaining wall design criteria;

l. Potential expansion or settlement risks; and

m. Pavement design sections.

We anticipate that a conceptual project layout will be available for our reference prior to our investigation. We would be able to incorporate a site plan into our report, if one is available in AutoCAD™ Release 14 or AutoCAD™ 2000 format.

We will mark our proposed exploratory trench locations and contact Underground Service Alert prior to our investigation. Although we will use reasonable caution during excavation of our exploratory trenches, Holdrege & Kull and our excavation subcontractor cannot be responsible for damage to utilities on the property that were not marked or were improperly marked by the Underground Service Alert or the property owner prior to our investigation.

Alternate Scope of Services

We understand that a geotechnical engineering investigation and report may also be required for the proposed Placer County Land Development Building (Project 4630), which is also to be located at the Dewitt Center. Under our alternate scope of services, the scope of services outlined above for the Auburn Justice Center would also be provided for the Land Development Building.

FEE

Our fee to complete the scope of services described above for the Auburn Justice Center site would be . We would be able to complete geotechnical investigations and reports for both the Auburn Justice Center and the Land Development Building sites, as described above for our alternate scope of services, for a fee of . We understand that the two investigations would be billed as separate projects. We will provide an operated backhoe to perform the investigation(s). Progress billing will be monthly using the attached 2002 fee schedule.

TIMING

We will be able to commence work on the project within two weeks of receiving authorization to proceed, weather permitting. Our geotechnical report(s) will be submitted within three weeks following completion of field work.

Thank you for the opportunity to provide geotechnical engineering services for your projects. Please call if you have any questions regarding our proposed scope of services. If this proposal is acceptable, please contact us.

Sincerely,

HOLDREGE & KULL

Jason Muir, P.E.
Project Engineer

enclosed: 2002 Fee Schedule

J:\WPDOCS\PRO\GTK\AUBURN JUSTICE CENTER.GTK

APPENDIX B *IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL ENGINEERING REPORT*

(included with permission of ASFE, Copyright 1992)

Important Information About Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. *No one except you* should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one—not even you*—should apply the report for any purpose or project except the one originally contemplated.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, *do not rely on a geotechnical engineering report* that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions *only* at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an *opinion* about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final*, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject To Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce such risks, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations", many of these provisions indicate where geotechnical engineers responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Rely on Your Geotechnical Engineer for Additional Assistance

Membership in ASFE exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

ASFE PROFESSIONAL FIRMS PRACTICING IN THE GEOSCIENCES

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APPENDIX C EXPLORATORY TRENCH LOGS

TRENCH T-1

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	TRENCH NO.
1751-01	AUBURN JUSTICE CENTER	1405 FEET MSL	10/18/02	1 OF 1	T-1

EXCAVATION METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CASE 580 BACKHOE	SLIDE HAMMER	NONE	NONE

SAMPLE NO.	BLOW COUNTS (N)	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
					GW	GRAVEL, UPPER 3 INCHES
CB 1-1	--	--	--	1		
BT 1-1	--	98.3	6.1		ML	DARK RED BROWN, DRY TO SLIGHTLY MOIST, MEDIUM DENSE SANDY SILT
				2		
				3	SM	COMPLETELY TO SEVERELY WEATHERED METAVOLCANIC ROCK (EXCAVATES AS LIGHT BROWN, SLIGHTLY MOIST SILTY SAND WITH ABUNDANT GRAVEL AND ANGULAR ROCK TO 6 INCHES IN DIAMETER)
				4		TRENCH TERMINATED AT 4 FEET BGS
				5		
				6		
				7		
				8		
				9		
				10		
				11		
				12		
				13		
				14		
				15		
				16		
				17		
				18		
				19		
				20		

TRENCH T-2

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	TRENCH NO.
1751-01	AUBURN JUSTICE CENTER	1410 FEET MSL	10/18/02	1 OF 1	T-2

EXCAVATION METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CASE 580 BACKHOE	HAND	NONE	NONE

SAMPLE NO.	BLOW COUNTS (N)	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
CB 2-1	--	--	--	1	ML	ORANGE BROWN, DRY TO SLIGHTLY MOIST, MEDIUM DENSE SANDY SILT
				2	SM	COMPLETELY TO MODERATELY WEATHERED, FRACTURED METAVOLCANIC ROCK (EXCAVATES AS LIGHT BROWN, SLIGHTLY MOIST, DENSE SILTY SAND WITH ABUNDANT ANGULAR ROCK)
				3		TRENCH TERMINATED AT 2.5 FEET BGS
				4		
				5		
				6		
				7		
				8		
				9		
				10		
				11		
				12		
				13		
				14		
				15		
				16		
				17		
				18		
				19		
				20		

TRENCH T-3

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	TRENCH NO.
1751-01	AUBURN JUSTICE CENTER	1421 FEET MSL	10/18/02	1 OF 1	T-3

EXCAVATION METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CASE 580 BACKHOE	NONE	NONE	NONE

SAMPLE NO.	BLOW COUNTS (N)	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
				0–2.5	SM	FILL: BROWN, DRY TO SLIGHTLY MOIST, MEDIUM DENSE SILTY SAND WITH ABUNDANT ANGULAR GRAVEL AND ANGULAR ROCK TO 12 INCHES IN DIAMETER
				2.5–4	RX	COMPLETELY TO MODERATELY WEATHERED METAVOLCANIC ROCK DENSITY INCREASES AT 4 FEET BGS - SLOW EXCAVATION
				4		TRENCH TERMINATED AT 4 FEET BGS

TRENCH T-4

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	TRENCH NO.
1751-01	AUBURN JUSTICE CENTER	1414 FEET MSL	10/18/02	1 OF 1	T-4

EXCAVATION METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CASE 580 BACKHOE	HAND	NONE	NONE

SAMPLE NO.	BLOW COUNTS (N)	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
					SM	FILL: ORANGE BROWN, SLIGHTLY MOIST, MEDIUM DENSE SILTY SAND WITH ABUNDANT ANGULAR ROCK AND TRACE CONCRETE AND ASPHALT
CB 4-1	--	--	--	1		
				2	SM	COMPLETELY TO MODERATELY WEATHERED, FRACTURED METAVOLCANIC ROCK (EXCAVATES AS ORANGE BROWN, SLIGHT MOIST, MEDIUM DENSE TO DENSE SILTY SAND WITH ABUNDANT ANGULAR ROCK TO 12 INCHES IN DIAMETER)
				3		TRENCH TERMINATED AT 2.5 FEET BGS
				4		
				5		
				6		
				7		
				8		
				9		
				10		
				11		
				12		
				13		
				14		
				15		
				16		
				17		
				18		
				19		
				20		

TRENCH T-5

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	TRENCH NO.
1751-01	AUBURN JUSTICE CENTER	1416 FEET MSL	10/18/02	1 OF 1	T-5

EXCAVATION METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CASE 580 BACKHOE	NONE	NONE	NONE

SAMPLE NO.	BLOW COUNTS (N)	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
CB 5-1	--	--	--	0–1.5	ML	RED BROWN TO ORANGE BROWN, SLIGHTLY MOIST, LOOSE TO MEDIUM DENSE SANDY SILT WITH MINOR GRAVEL
				1.5–4	SM	COMPLETELY TO SEVERELY WEATHERED METAVOLCANIC ROCK (EXCAVATES AS LIGHT BROWN, SLIGHTLY MOIST, MEDIUM DENSE TO DENSE SILTY SAND AND SANDY SILT WITH COMMON ANGULAR GRAVEL) DENSITY INCREASES AT 4 FEET BGS – SLOW EXCAVATION
				4		TRENCH TERMINATED AT 4 FEET BGS

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TRENCH T-6

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	TRENCH NO.
1751-01	AUBURN JUSTICE CENTER	1411 FEET MSL	10/18/02	1 OF 1	T-6

EXCAVATION METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CASE 580 BACKHOE	SLIDE HAMMER	NONE	NONE

SAMPLE NO.	BLOW COUNTS (N)	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
				0–1	ML / SM	FILL: BROWN, DRY, LOOSE SANDY SILT AND SILTY SAND WITH ANGULAR GRAVEL
				1	RX	ASPHALT CONCRETE PAVEMENT ENCOUNTERED 1 FOOT BGS
CB 6-1	--	--	--	2–6	SM	FILL: BROWN, SLIGHTLY MOIST, DENSE SILTY SAND WITH ANGULAR GRAVEL AND MINOR CLAY TOO DENSE AND GRAVELLY TO SAMPLE WITH SLIDE HAMMER MOIST SOIL AT 5.5 FEET BGS
				6–8	CL	SEVERELY TO MODERATELY WEATHERED, FRACTURED METAVOLCANIC ROCK WITH OLIVE, MOIST, FIRM CLAY
				8		TRENCH TERMINATED AT 8 FEET BGS

Depth scale: 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20

TRENCH T-7

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	TRENCH NO.
1751-01	AUBURN JUSTICE CENTER	1411 FEET MSL	10/18/02	1 OF 1	T-7

EXCAVATION METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CASE 580 BACKHOE	NONE	NONE	NONE

SAMPLE NO.	BLOW COUNTS (N)	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
					ML	RED BROWN, DRY, MEDIUM DENSE SANDY SILT WITH ANGULAR ROCK
				1	RX	SEVERELY TO MODERATELY WEATHERED, FRACTURED METAVOLCANIC ROCK
				2		NEAR REFUSAL ON MODERATELY WEATHERED METAVOLCANIC ROCK AT 1.5 FEET BGS
				3		
				4		
				5		
				6		
				7		
				8		
				9		
				10		
				11		
				12		
				13		
				14		
				15		
				16		
				17		
				18		
				19		
				20		

TRENCH T-8

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	TRENCH NO.
1751-01	AUBURN JUSTICE CENTER	1413 FEET MSL	10/18/02	1 OF 1	T-8

EXCAVATION METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CASE 580 BACKHOE	NONE	NONE	0-3 FEET BGS

SAMPLE NO.	BLOW COUNTS (N)	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
				1		FILL: BROWN TO RED BROWN, SLIGHTLY MOIST, LOOSE SILTY SAND WITH ANGULAR ROCK
				2	SM	
				3		FILL: BROWN TO GRAY, SLIGHTLY MOIST TO MOIST, LOOSE SILTY SAND WITH ABUNDANT ORGANIC MATERIAL AND ANGULAR ROCK
				4	SM	
				5		FILL: BROWN TO RED BROWN, SLIGHTLY MOIST, LOOSE SILTY SAND WITH ANGULAR ROCK
				6	SM	
				7		
				8		NEAR REFUSAL ON NATIVE ROCK AT 8 FEET BGS
				9		
				10		
				11		
				12		
				13		
				14		
				15		
				16		
				17		
				18		
				19		
				20		

TRENCH T-9

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	TRENCH NO.
1751-01	AUBURN JUSTICE CENTER	1413 FEET MSL	10/18/02	1 OF 1	T-9

EXCAVATION METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CASE 580 BACKHOE	NONE	NONE	0-5 FEET BGS

SAMPLE NO.	BLOW COUNTS (N)	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
				0–7	SM	FILL: BROWN TO RED BROWN, SLIGHTLY MOIST, LOOSE SILTY SAND WITH ANGULAR ROCK AND MINOR CONCRETE AND ASPHALT
				7–8	ML	RED BROWN, SLIGHTLY MOIST, MEDIUM DENSE SANDY SILT
				8–9	SM	COMPLETELY TO SEVERELY WEATHERED METAVOLCANIC ROCK (EXCAVATES AS LIGHT BROWN, SLIGHTLY MOIST, MEDIUM DENSE SILTY SAND WITH ANGULAR ROCK TO 6 INCHES IN DIAMTER)
				9		TRENCH TERMINATED AT 9 FEET BGS

Depth scale (ft): 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20

TRENCH T-10

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	TRENCH NO.
1751-01	AUBURN JUSTICE CENTER	1412 FEET MSL	10/18/02	1 OF 1	T-10

EXCAVATION METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CASE 580 BACKHOE	NONE	NONE	NONE

SAMPLE NO.	BLOW COUNTS (N)	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
				1	SM	FILL: BROWN, DRY TO SLIGHTLY MOIST, LOOSE SILTY SAND WITH ANGULAR ROCK TO 12 INCHES IN DIAMETER
				2	RX	SEVERELY TO MODERATELY WEATHERED METAVOLCANIC ROCK
				3		TRENCH TERMINATED AT 3 FEET BGS
				4		
				5		
				6		
				7		
				8		
				9		
				10		
				11		
				12		
				13		
				14		
				15		
				16		
				17		
				18		
				19		
				20		

TRENCH T-11

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	TRENCH NO.
1751-01	AUBURN JUSTICE CENTER	1405 FEET MSL	10/22/02	1 OF 1	T-11

EXCAVATION METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CASE 580 BACKHOE	HAND	NONE	NONE

SAMPLE NO.	BLOW COUNTS (N)	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)		USCS	DESCRIPTIONS/REMARKS
				0–1		ML	DARK RED BROWN, DRY, MEDIUM DENSE SANDY SILT WITH MINOR CLAY AND ABUNDANT ANGULAR ROCK
				1–2.5		RX	MODERATELY WEATHERED, FRACTURED METAVOLCANIC ROCK (DIFFICULT EXCAVATION)
							TRENCH TERMINATED AT 2.5 FEET BGS

DEPTH SCALE: 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20

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TRENCH T-12

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	TRENCH NO.
1751-01	AUBURN JUSTICE CENTER	1410 FEET MSL	10/22/02	1 OF 1	T-12

EXCAVATION METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CASE 580 BACKHOE	NONE	NONE	MINOR 0-8 FEET BGS

SAMPLE NO.	BLOW COUNTS (N)	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
				0–8	SM	FILL: RED BROWN, DRY TO SLIGHTLY MOIST, LOOSE TO MEDIUM DENSE SILTY SAND WITH ABUNDANT ANGULAR ROCK TO 6 INCHES IN DIAMETER AND MINOR ASPHALT MINOR WOOD DEBRIS ENCOUNTERED AT 8 FEET BGS
				8–8.5	CL	OLIVE, MOIST, SOFT, SILTY CLAY WITH ANGULAR ROCK
						TRENCH TERMINATED AT 8.5 FEET BGS (DIFFICULT EXCAVATION, NO ROCK)

Depth scale: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20

HOLDREGE & KULL

TRENCH T-13

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	TRENCH NO.
1751-01	AUBURN JUSTICE CENTER	1408 FEET MSL	10/22/02	1 OF 1	T-13

EXCAVATION METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CASE 580 BACKHOE	NONE	NONE	MINOR 0-6 FEET BGS

SAMPLE NO.	BLOW COUNTS (N)	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
				0-6	SM	FILL: RED BROWN, DRY TO SLIGHTLY MOIST, LOOSE TO MEDIUM DENSE SILTY SAND WITH ANGULAR ROCK TO 9 INCHES IN DIAMETER AND TRACE DEBRIS (NYLON ROPE, PLASTIC BAG)
				6-8	ML	DARK RED BROWN, SLIGHTLY MOIST, MEDIUM DENSE SANDY SILT WITH MINOR CLAY
				8		TRENCH TERMINATED AT 8 FEET BGS

HOLDREGE & KULL

TRENCH T-14

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	TRENCH NO.
1751-01	AUBURN JUSTICE CENTER	1408 FEET MSL	10/22/02	1 OF 1	T-14

EXCAVATION METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CASE 580 BACKHOE	NONE	NONE	NONE

SAMPLE NO.	BLOW COUNTS (N)	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
CB 14-1	--	--	--	1	SM	RED BROWN, DRY TO SLIGHTLY MOIST, MEDIUM DENSE SANDY SILT AND SILTY SAND WITH ANGULAR ROCK TO 6 INCHES IN DIAMETER
BT 14-1	--	112.3	6.1			
BT 14-2	--	104.1	9.6	2		
					RX	MODERATELY WEATHERED, FRACTURED METAVOLCANIC ROCK
				3		NEAR REFUSAL AT 3 FEET BGS ON RESISTANT ROCK
				4		
				5		
				6		
				7		
				8		
				9		
				10		
				11		
				12		
				13		
				14		
				15		
				16		
				17		
				18		
				19		
				20		

TRENCH T-15

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	TRENCH NO.
1751-01	AUBURN JUSTICE CENTER	1412 FEET MSL	10/22/02	1 OF 1	T-15

EXCAVATION METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CASE 580 BACKHOE	NONE	NONE	NONE

SAMPLE NO.	BLOW COUNTS (N)	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
				0–1	SM	FILL: BROWN, DRY, LOOSE TO MEDIUM DENSE SILTY SAND WITH ABUNDANT ROCK
				1–2.5	RX	COMPLETELY TO MODERATELY WEATHERED METAVOLCANIC ROCK
				2.5		TRENCH TERMINATED AT 2.5 FEET BGS

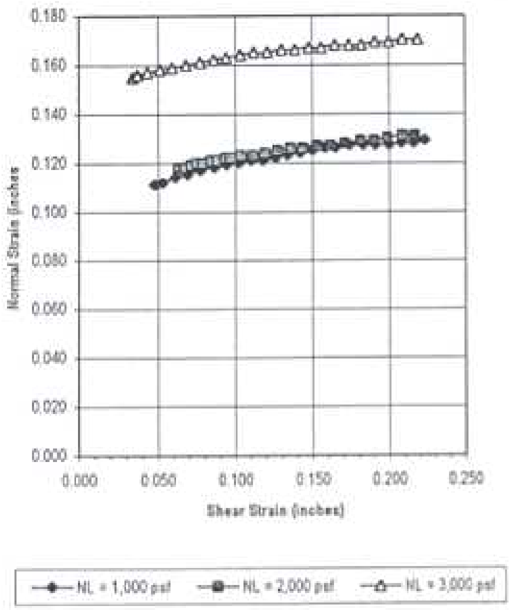
Depth scale (ft): 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20

APPENDIX D LABORATORY TEST DATA

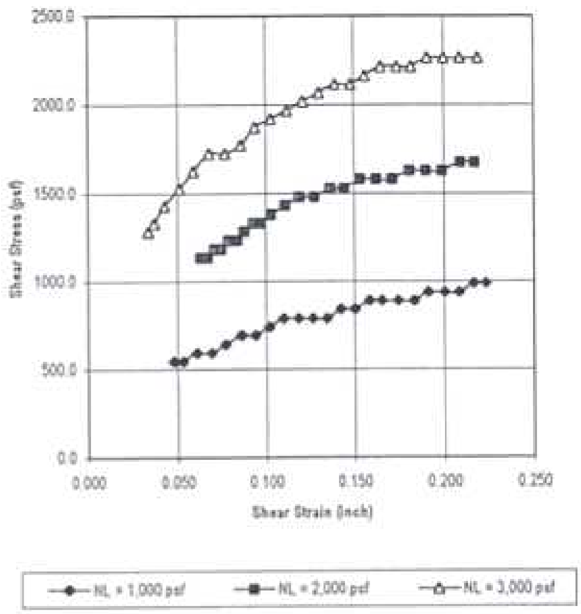
DIRECT SHEAR

TEST RESULTS

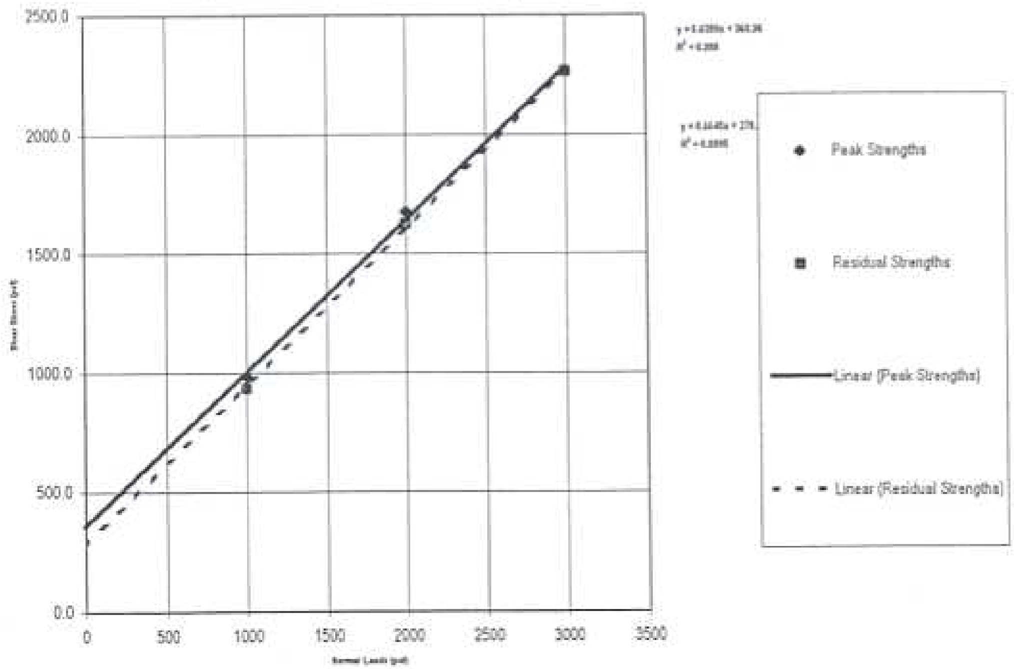
Shear Strain vs. Normal Strain



Shear Strain vs. Shear Stress



Mohr-Columb Failure Envelope



SHEAR STRENGTH TEST RESULTS		
PARAMETERS	PEAK STRENGTH	RESIDUAL STRENGTH:
FRICTION ANGLE, (degree)	32.6	33.6
COHESION, (psf)	361.0	279.0

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PROJECT NAME:	Auburn Justice Center		
PROJECT NO.:	1751-01	DATE:	10/24/02
BORING / TRENCH NO.:	T-14	LAB NO.	2-552
SAMPLE NO.:	BT 14-1	SAMPLE DEPTH (ft):	1.0
DESCRIPTION:	Red-Brown Sandy Silt with Gravel		

COMPACTION TEST

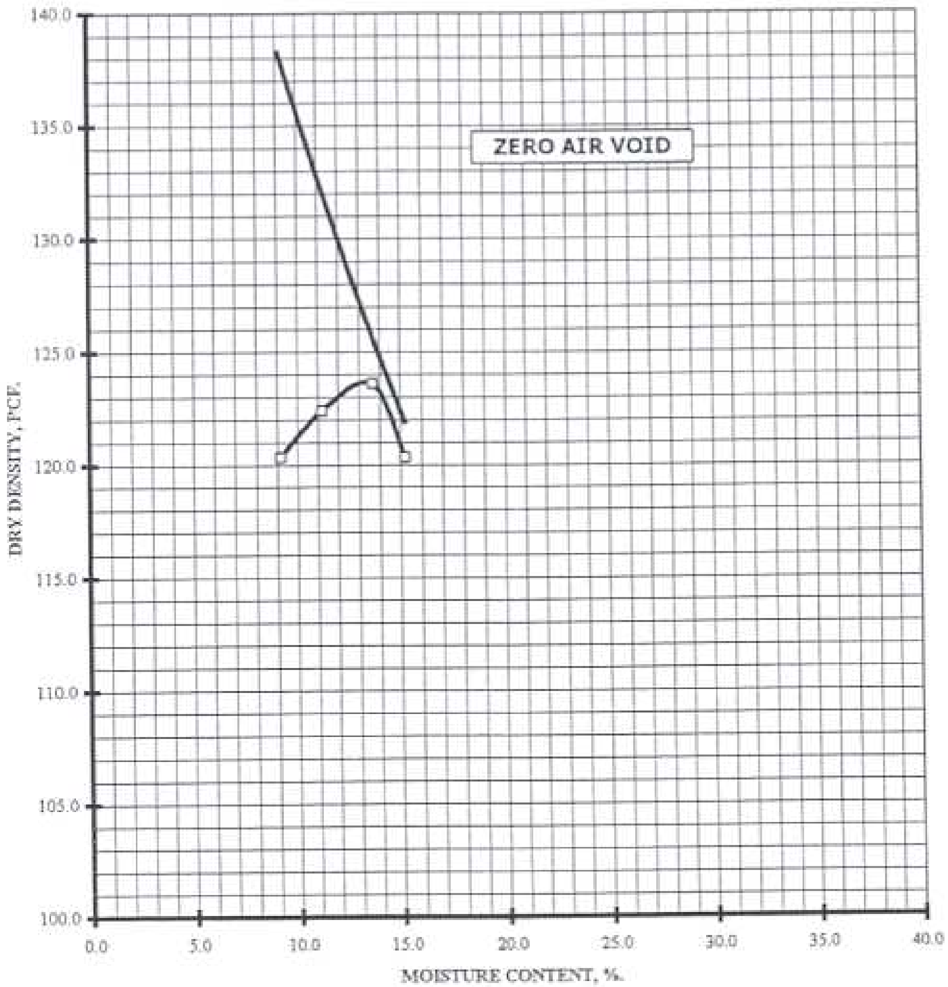
☐ ASTM D698 ■ ASTM D1557

Project No.:	1751-01	Project Name	Auburn Justice Center	Date:	10/23/02
Sample No.:	CB 6-1	Boring/Trenc T-6	Depth, ft.: 1.5-4.0	Tested By:	JCS
Description:	Brown Silty Sand with Angular Gravel			Checked By:	MLH
Sample Location:				Lab. No.:	2-552

Vol., Mold, cf.:	0.03330	No. of Layers:	5	Hammer Drop:	18
Method:	B	Blows/Layer:	25	Hammer Weight:	10

Trial Number		+4	+6	+8	+10	
Container Number		1	2	3	S4	
Wet Soil + Container	(gms.)	1023.40	1271.40	1102.30	1117.20	
Dry Soil + Container	(gms.)	950.20	1159.70	988.20	990.20	
Container Weight	(gms.)	151.70	163.40	153.60	155.60	
Weight of Water	(gms.)	73.20	111.70	114.10	127.00	
Weight of Dry Soil	(gms.)	798.50	996.30	834.60	834.60	
Moisture Content	(%)	9.2	11.2	13.7	15.2	
Wet Soil + Mold	(gms.)	4010	4082	4148	4120	
Weight of Mold	(gms.)	2026	2026	2026	2026	
Wet Weight of Soil	(lbs.)	4.37	4.53	4.68	4.62	
Wet Unit Weight	(pcf.)	131.3	136.1	140.5	138.6	
Dry Unit Weight	(pcf.)	120.3	122.4	123.6	120.3	

% Rock Retained					
3/8" sieve	3/4" sieve	Maximum Dry Density, pcf.:	124.0	Max. w/ Rock Correction	124.0
		Optimum Moisture Content:	15.0	Adj. Optimum Moisture %	15.0
		Est. Specific Gravity:	2.78	Est. Specific Gravity:	2.78

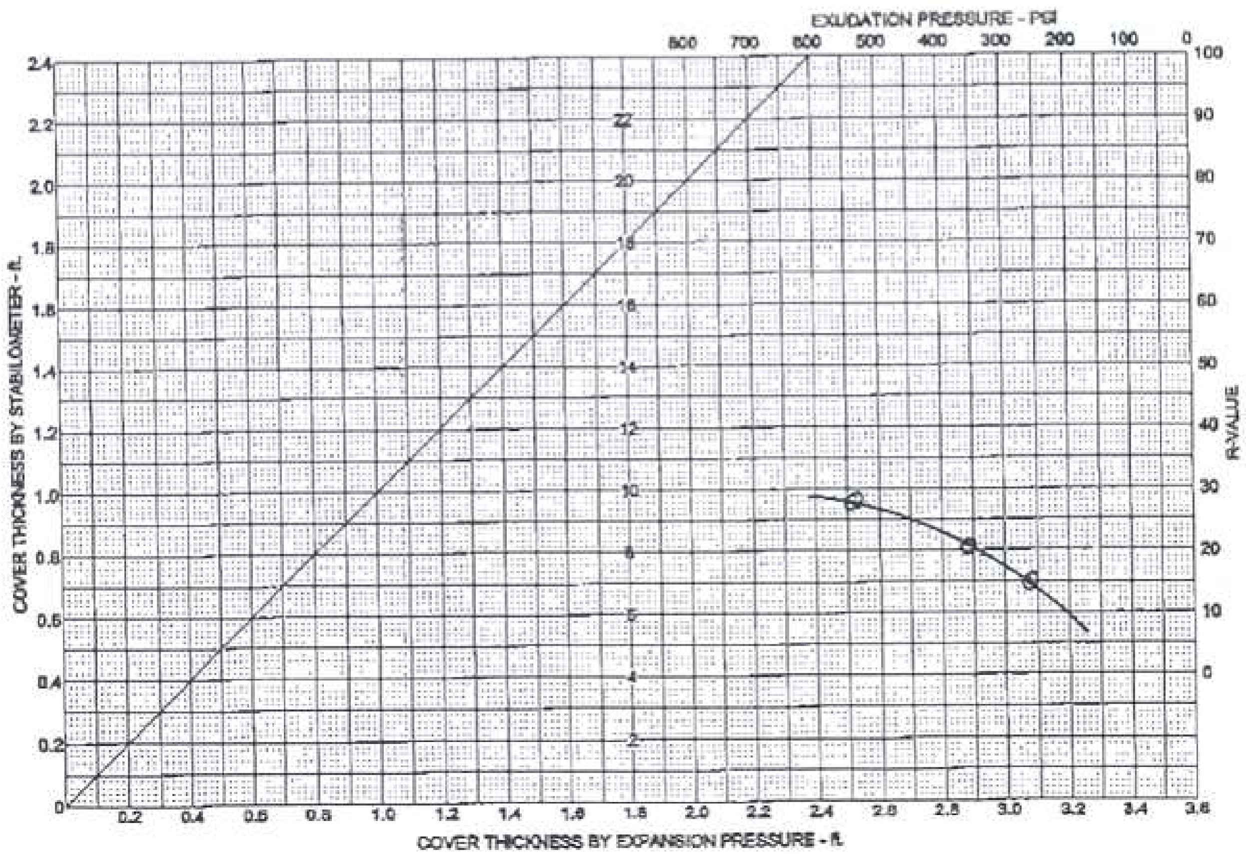


HOLDREGE & KULL

RESISTANCE VALUE

CALTRANS 301

SPECIMEN	A	B	C
EXUDATION PRESSURE, PSI	263	358	541
EXPANSION DIAL (.0001")	11	19	56
EXPANSION PRESSURE, PSF	48	82	242
RESISTANCE VALUE, R	15	21	28
% MOISTURE AT TEST	16.4	15.5	14.6
DRY DENSITY AT TEST, PCF	115	116	118
R VALUE @ 300 PSI EXUDATION PRESSURE		17	
R VALUE BY EXPANSION PRESSURE (TI=)			



WALLACE · KUHL & ASSOCIATES INC.	HOLDREGE AND KULL	WKA NO:	4848.01
	Holdrege and Kull Job Name: Auburn Justice Center	DATE:	10/28/02
	Sample Comp-1; Reddish brown rocky clayey silt	LAB NO:	9807